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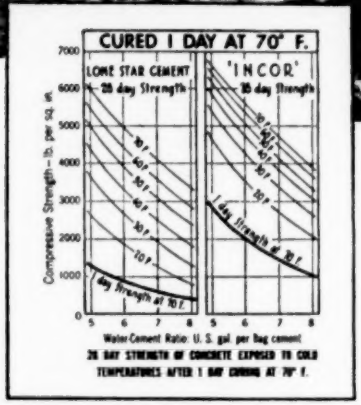

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Vol. 26—No. 3 November 1954 (Proceedings Vol. 51)

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The ACI JOURNAL technical pages are a medium for reporting what is of current interest in the concrete field; in research, design, construction, maintenance, or manufacturing. Members and readers are urged to "have their say" as a means of sharing knowledge and experience; time and changing interests demand that new names and new ideas take their places beside the old. Contributions describing new materials and techniques or new applications of familiar practices are welcomed by the Institute.

To guide the flow of material the Board of Direction has assigned to the Technical Activities Committee the selection of papers, committee reports, discussions, and other contributions for JOURNAL publication. To assist in their technical appraisal a corps of experts in various segments of the field volunteer their services. From the technical standpoint, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

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To expedite appraisal and publication, if accepted, contributors are requested to furnish glossy prints or inked tracings of illustrations accompanying manuscripts submitted for consideration. Further details of form and physical arrangement of contributions begin on p. 14, 1953 *ACI Directory*. Separate prints of the "publications policy" are available free upon request to the Secretary.

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In ACI Journal next month

"Shear Strength of Reinforced Concrete Beams. Part 1—Tests of Simple Beams," by K. G. Moody, I. M. Viest, R. C. Elstner, and E. Hognestad, is a progress report giving the results of shear tests performed on 40 simple beams without web reinforcement and two with reinforcement.

"The Effect of Algae Infested Water on the Strength of Concrete," by Benjamin C. Doell, discusses a study to establish a relationship between loss in strength of concrete and various concentrations of organic matter contained in algae infested water. Various amounts were added to the mixing water of test batches, with the resulting cylinders tested at 28 and 60 days. A close approximation of the allowable concentration of organic matter in a good mixing water is established.

Paul W. Abeles presents "Static and Fatigue Tests on Partially Prestressed Concrete Constructions" in which the essential features of partial prestressing are investigated. Static and fatigue tests as well as routine acceptance tests are described because they are important to show the essential properties of prestressed concrete. A survey of the practical application is given with regard to bridge and roof constructions and to designs of mast structures. In an appendix the basic formulas are presented and some examples shown.

During the past 10 or 15 years many investigations have been carried out on the question of durability of concrete under freezing and thawing. A survey and analysis of the more important investigations on the subject shows, however, that significant general conclusions can be drawn. In "Observations on the Resistance of Concrete to Freezing and Thawing," Hubert Woods presents some of these conclusions in a manner designed to show their reasonableness, but without detailed evidence. It is a concise presentation which the reader can check against published data.

In the paper "Some Failures of Reinforced Concrete Storage Bins," Louis E. Vandergrift describes some investigational work done in connection with the structural failure of some reinforced concrete grain storage bins. He illustrates various failures in the field and describes some tests of models. He notes that the failures which have come to his attention, with the exception of one, were due to incorrect provision for reinforcement at critical points.

Title No. 51-9

Model Analysis of a Skewed Rigid Frame Bridge and Slab*

By D. H. PLETTA and D. FREDERICK†

SYNOPSIS

Results of an experimental analysis of a small aluminum model of a skewed-slab rigid-frame bridge are presented. A model theory is also checked using reinforced concrete as the model material by testing three sizes of geometrically similar simple beams and a model of a skewed slab with curbs. The slab is a $\frac{1}{8}$-scale model similar to a $\frac{1}{2}$-scale model tested previously at the University of Illinois. Results indicate that small reinforced concrete models do behave just as their prototypes up to the ultimate load, and it now seems possible to test models of entire structures rather than just their component parts as has been the practice in the past.

INTRODUCTION

Skewed-slab rigid-frame bridges, like most other structures, have had a long satisfactory service record; yet there has been little experimental verification of the assumptions on which their designs were based. This paper describes tests on an aluminum model of a skewed-slab rigid-frame bridge, some small reinforced concrete beams and cylinders, and a skewed reinforced concrete slab. The program is only partially complete and will include a test of a reinforced concrete model of a skewed-slab rigid-frame bridge before it is concluded. The research was undertaken not only to determine the behavior of structures such as this but to see whether practical results could be obtained from small models made of reinforced concrete.

ALUMINUM MODEL

Structural analysis is generally based on the theory of elasticity. Unfortunately, no known analytical solution exists for skewed-slab rigid frames, nor even for a skewed slab with two free edges. It was felt that the solution of the governing differential equation might be simplified if the variation of thrust, shear, and moment along the knee of the frame and deflection along the free edge of the slab were known. Such data were obtained from the small aluminum model shown in Fig. 1 and 2. It was fabricated by the Aluminum Company of America of $\frac{1}{2}$-in. rolled 24S plate, welded along the knee by a semi-automatic, inert-gas metal arc employing $\frac{1}{16}$-in. diameter 43S filler wire, 250 amp, d-c and 50 cu ft per hr of argon gas flow. A V joint was used and filled with two passes of the arc on the outside and one on the inside to provide the fillet. The model was solution heat treated to T 4 following the welding by heating to 920 F, holding for 1 hr, and water quenching.

*Presented as part of the work of ACI Committee 314, Rigid Frame Bridges, at the ACI 50th annual convention, Denver, Colo., Feb. 24, 1954. Title No. 51-9 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 3, Nov. 1954, *Proceedings*, V. 51. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1955. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Members American Concrete Institute, Professor and Head of Applied Mechanics, and Research Associate Professor of Applied Mechanics, respectively, Virginia Polytechnic Institute, Blacksburg, Va.

TABLE 1—PHYSICAL PROPERTIES OF ALUMINUM SAMPLES

Specimen no.	Direction of rolling	Proportional limit, psi	Ultimate strength, psi	Modulus of elasticity, psi		Percent elongation in 2 in.	Poisson's ratio
				Dial gage	SR-4 gage		
1	Parallel	32,300	68,750	10,100,000	——	22.0	——
2	Parallel	32,400	68,600	10,450,000	10,950,000	19.5	0.321
3	Perpendicular	29,650	66,800	10,460,000	——	12.5*	——
4	Perpendicular	31,600	66,900	10,460,000	10,920,000	20.0	0.319

*Failure outside gage marks.

Physical properties of the aluminum were tested from 0.5 in. square tensile test coupons cut from the model. Two specimens were taken parallel and two perpendicular to the direction of rolling of the original plate. These results are listed in Table 1.

The remarkable consistency of these results justified the assumptions made in later calculations regarding the isotropic nature of the model material, and they stand as mute evidence to the excellent workmanship of the Aluminum Company of America's research laboratories.

The location of all SR-4 (type A-7) gages, nine deflection stations, and nine loading points are shown in Fig. 1. The concentrated load was applied to the deck through a short cylinder ¾ in. in diameter in 500-lb increments up to a maximum of 1575 lb starting with an initial load of 75 lb as shown in Fig. 2. The initial load was used to insure complete seating of the knife edges at the bottom of each abutment in the V-grooved bases. This base was provided with leveling screws and set in a thin plaster of paris bed. Lateral separation of the hinged supports was prevented by tying the two bases together with steel straps. Sidesway of the frame was prevented by friction between the loading head and cylinder.

The model was also loaded with uniform loads of 3, 6, and 9 psi applied over the deck surface through an inflated rubber air cell. This air cell was constrained in a wood casing

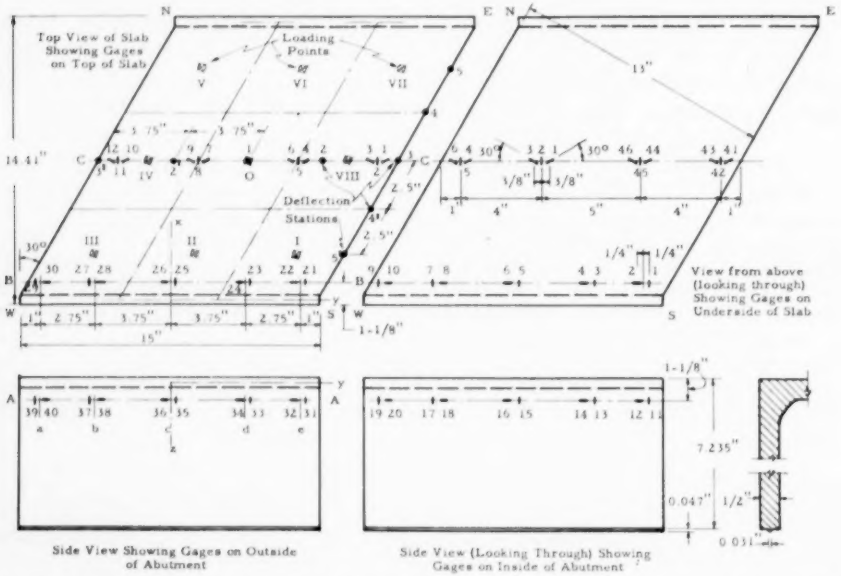
Top View of Slab Showing Gages on Top of Slab
Loading Points
Deflection Stations
View from above (looking through) Showing Gages on Underside of Slab
Side View Showing Gages on Outside of Abutment
Side View (Looking Through) Showing Gages on Inside of Abutment

Fig. 1—Location of strain gages, deflection stations, and loading points on the aluminum model

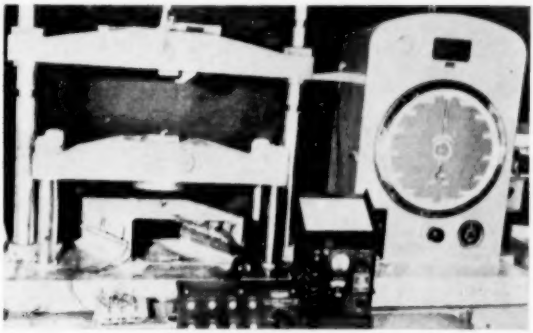

Fig. 2—Testing arrangement for the aluminum model

whose shape coincided in plan with that of the skewed deck except for a small clearance. Slow compression of the air in the cell did not produce any significant variation of temperature as measured on a thermometer reading to 0.05 C. Hence, no correction in recorded strains from gages on the deck was necessary.

RESULTS FOR THE ALUMINUM MODEL

Fig. 1 indicates that individual type A-7 gages were cemented in clusters on axes parallel to the knee at the centerline (*C-C*) of the deck, and near the knee in the deck (*B-B*) and abutment (*A-A*). This cluster arrangement was used instead of the more familiar triangular rosette pattern so that more precise strains at a given point on a single axis could be found by interpolation. Curves were plotted for the gage readings versus their position along these axes, and the (point) strains at selected intervals then read from the curves. Fig. 3 illustrates this technique for gages along *C-C* for a 1500-lb load in panel 0.

Three such point strains were converted to principal stresses at four points 1 in. and 5 in. from the free edges of the deck along line *C-C* by nomographic charts.[1] These principal stresses and their directions on both the top and bottom surfaces of the deck are plotted on influence diagrams for these four points in Fig. 4 and 5. A study of these diagrams will reveal that the results agree fairly well with each other when compared on the basis of superposition, as for instance, those for the stresses at 2 *CW* due to a load in panel 0 (Fig. 4a)

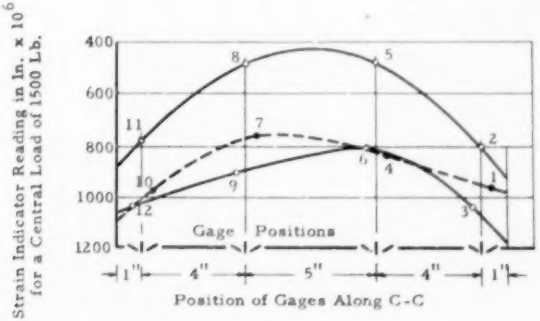

Fig. 3—Relation between strain and position of gage along line C-C for 1500-lb load

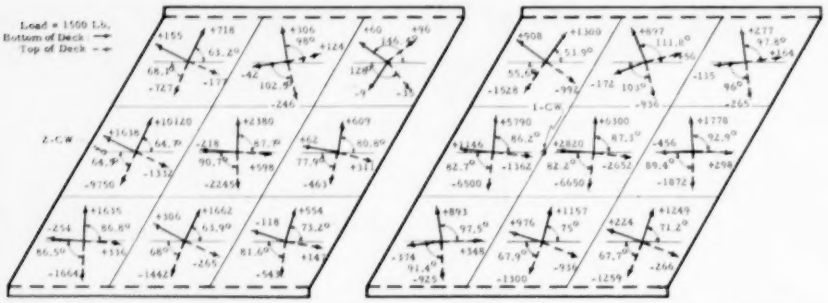

Fig. 4a (left)—Influence diagram for principal stress (psi) at point 2-CW. Fig. 4b (right)—Influence diagram for principle stress (psi) at 1-CW

and at 2 *CE* due to a load in panel 0 (Fig. 5b). The deviations from exact agreement are probably due to imperfect seating.

The magnitude and orientation of the principal stresses at the same four points along *C-C* for a uniform load of 9 psi are shown in Fig. 6. The maximum error here is of the order of approximately 5 percent.

Along the axes *A-A* and *B-B* near the knee, where the gages were oriented to read ϵ_x (or ϵ_i) and ϵ_y directly, such (point) strains were converted to stresses σ_x (or σ_i) and σ_y for both the outside (*o*) and inside (*i*) of the deck and abutment by substituting in

$$\sigma_x = \frac{E}{1-\nu^2}(\epsilon_x + \nu\epsilon_y) \text{ and } \sigma_y = \frac{E}{1-\nu^2}(\epsilon_y + \nu\epsilon_x) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and letting $E = 10{,}360{,}000$ psi and $\nu = 0.32$. Curves for σ_x and σ_z versus their respective positions along *B-B* and *A-A* respectively are plotted in Fig. 7 and 8.

A check on the accuracy of the results was made by comparing the total thrust in each abutment to the total load. The stress was assumed to vary linearly with the plate thickness at any point. Hence, the normal and bending stresses may be expressed as

$$\sigma_{zo} = \frac{\sigma_{zi} + \sigma_{zo}}{2} = \text{Normal stress due to thrust in the abutment}$$

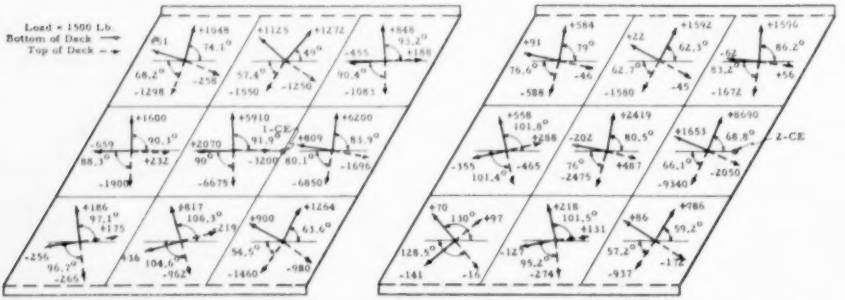

Fig. 5a (left)—Influence diagram for principal stress (psi) at 1-CE. Fig. 5b (right)—Influence diagram for principal stress (psi) at 2-CE

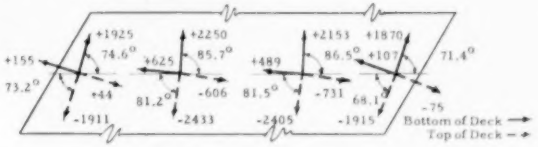

Fig. 6—Variation of principal stress (psi) along section C-C for 9-psi uniform load

and $\sigma_{zm} = \dfrac{\sigma_{zi} - \sigma_{zo}}{2}$ = Bending stress due to moment in the abutment

Similar expressions can be written for σ_{zn} and σ_{zm} in the deck. It was further assumed that symmetry exists since the thrust for a load in panel I in one abutment would be the same as the thrust in the other abutment due to a load in panel V. To satisfy statics the total thrust

$$P = N_z + N_z' = t\int \sigma_{zn}\, dy + t\int \sigma'_{zn}\, dy \text{ (where } t = \text{plate thickness)}$$

Unfortunately such a simple relationship failed to check by the percentages given in Table 2.

It was felt that the proximity of the inside gages along lines *A-A* and *B-B* to the fillet resulted in localized increases in the strains. Assuming a constant stress-concentration factor K to exist at these inner points, statics would be satisfied if

$$P = t\int \frac{\sigma_{zo} + K\sigma_{zi}}{2}\, dy + t\int \frac{\sigma'_{zo} + K\sigma'_{zi}}{2}\, dy$$

Recalculation for the K factor as shown in Table 2 indicated a variation of ± 3.96 percent. Hence, it was felt that the curves of Fig. 7 and 8 could be used if increased by their respective factors.

The variation in shears and deck moment near the knee can likewise be

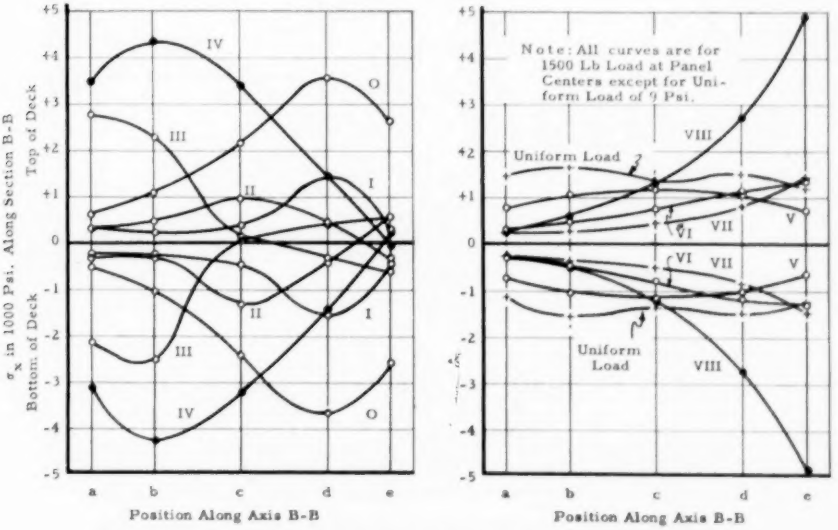

Fig. 7—Relation between unit stress and position of gages along line *B-B*

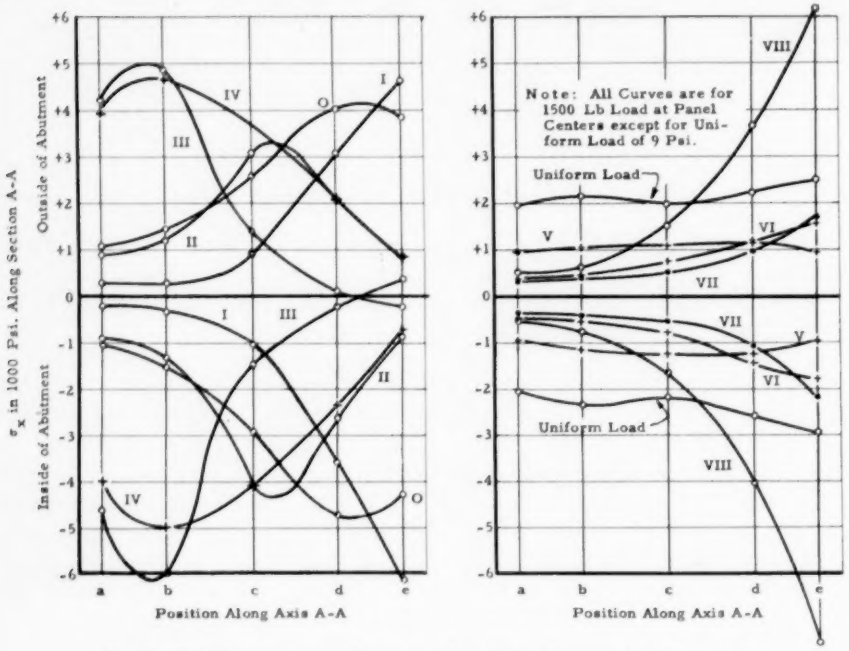
Outside of Abutment
Inside of Abutment
σ_x in 1000 Psi. Along Section A-A
Position Along Axis A-A
Note: All Curves are for 1500 Lb Load at Panel Centers except for Uniform Load of 9 Psi.
Uniform Load
Uniform Load
Position Along Axis A-A

Fig. 8—Relation between unit stress and position of gages along line A-A

obtained from the curves of Fig. 7 and 8 by satisfying the laws of statics for the free body diagram of Fig. 9. Then

$$N_z - V_z = 0$$

$$N_x - V_x = 0$$

$$M_{yD} + \frac{7}{8}N_z - \frac{7}{8}V_x - M_{yA} = 0$$

where $$N_z = t\int \frac{\sigma_{zo} + K\sigma_{zi}}{2}\,dy$$

and $$M_{yD} = \frac{t^2}{6}\int \frac{K'\sigma_{zi} - \sigma_{zo}}{2}\,dy$$

and $$M_{yA} = \frac{t^2}{6}\int \frac{K\,\sigma_{zi} - \sigma_{zo}}{2}\,dy$$

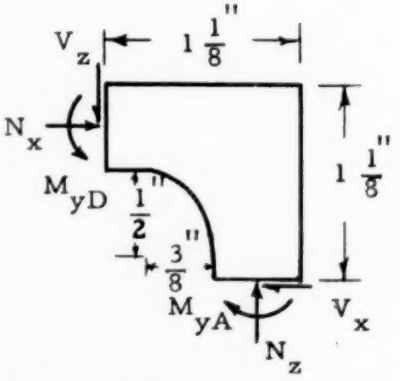

Fig. 9—Free body diagram showing shears and moments at knee

Space does not permit inclusion of the results here.

The deflections of one free edge and of the centerline of the deck are plotted in Fig. 10 for the 1500-lb load in each of the nine panels, and for the uniform load of 9 psi.

TABLE 2—COMPARISON OF PANEL LOADS

	Loaded panel					Uniform load
	0	I	II	III	IV	
Percent difference	80.0	26.7	47.8	26.7	57.5	7.0
Stress factor	1.100	1.040	1.071	1.060	1.064	1.017

MODEL REINFORCED CONCRETE BEAM AND CYLINDER TESTS

It was thought advisable to develop the technique of making small models of reinforced concrete and to check the application of model analysis to structures made of such composite material by starting with simple beams. Theoretically, models made of identical materials will behave in similar fashion even in the nonlinear range as the loading approaches the ultimate provided the material is isotropic, homogeneous, and continuous and the concentrated loads vary as the square of the scale ratio. It is impossible to scale down any of the ingredients of a concrete mix except the maximum size of the aggregate. Hence, there may be some doubt about the behavior of models made of concrete.

The beams used in these tests were made in three sizes whose scale ratio was chosen as 0.1, 0.2, and 0.3 of the prototype. The smallest beam was 1⅝ in. wide, 2¼ in. high, and 24 in. long, supported on a 21.33-in. span

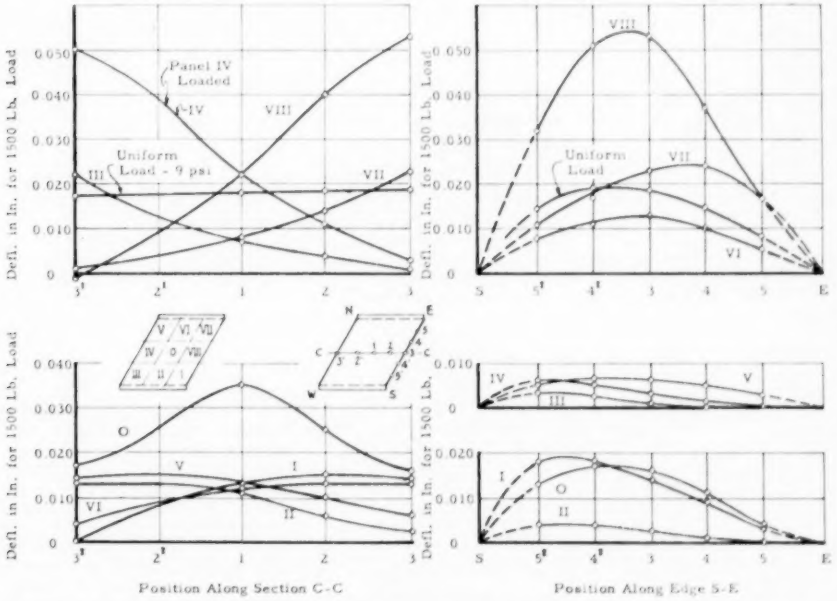

Fig. 10—Deflections of free edge S-E and centerline C-C for 1500-lb load at each of the nine panels and 9-psi uniform load

and loaded at the quarter-points. The reinforcing steel was $\frac{1}{8}$ in. in diameter and the maximum size of aggregate was $\frac{1}{4}$ in. All dimensions for the two larger beams were 2 or 3 times these, respectively. All three concretes had a W/C ratio by volume of 0.957 and a 6-in. slump.

Maximum loads on the beams were 400, 1600, and 3600 lb, respectively. Readings of the center deflection and strain in the top steel were taken during the tests. These results are plotted in Fig. 11a and 11b and indicate that the deflections do vary in the ratio 1:2:3 and that the strains for the gages attached before casting (old) are essentially the same, as they should be for perfect similarity. The agreement for the (new) gages attached to the reinforcing through holes drilled in the concrete after the initial tests were completed is not as good for it varies by ± 12.8 percent.

Three beams of each size and nine companion test cylinders were cast of the same concrete mix simultaneously and subjected to identical curing. Three of the nine cylinders were 2 in. in diameter and 4 in. high, three were 4 in. in diameter and 8 in. high, and three were 6 in. in diameter and 12 in. high. Similar sets of cylinders were used for each beam size so that data could be accumulated on the variation of the physical properties of the concrete with size of aggregate and the size of the cylinder. Fig. 11c and 11d indicate that there is a slight decrease in the modulus of elasticity and ultimate strength as the test specimen increases in size.

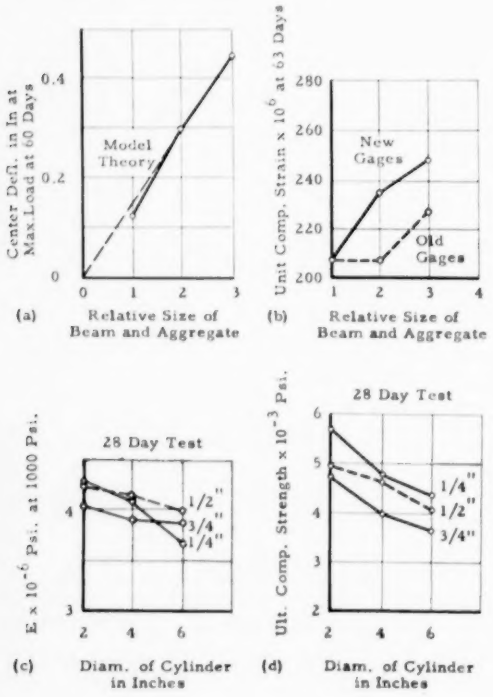

Fig. 11a (upper left)—Relation between center deflection and relative size of beam and aggregate. Fig. 11b (upper right)—Relation between unit compressive strain and relative size of beam and aggregate. Fig. 11c (lower left)—Relation of modulus of elasticity E and diameter of cylinder. Fig. 11d (lower right)—Relation between ultimate compressive strength and diameter of cylinder

Results for the test beams and cylinders are summarized here as a matter of convenience but have been described in much more detail elsewhere.[2]

MODEL SKEWED SLAB TEST

As a further check on the comparison of the behavior of small reinforced concrete models and their prototypes, a ⅛-scale reinforced concrete skewed slab was cast and tested. This slab was a model of Bridge M45-2 fabricated and tested on a ½-scale basis at the University of Illinois in 1949.[3] Every detail was scaled down by ¼. Details of its construction are shown in Fig. 12a and Fig. 13. Type A-12 SR-4 gages were cemented to the ⅛-in. diameter reinforcing and to the top and bottom concrete surfaces at selected points as indicated in Fig. 12b. These gages had a 1⅝-in. gage length which corresponded with that of the 6½-in. graphic strain gages used in the Illinois model. Fig. 12b also shows the points at which concentrated loads were applied through 1⅞-in. steel disks and a ⅜-in. rubber pad in contact with the slab.

The concrete was made with ⅛-in. maximum size aggregate with W/C = 1.00 by volume. It was cured for two weeks under wet burlap and had a 4¼-in. slump. Three companion test cylinders of each of three sizes were made at the same time and were tested at 28 days with the results indicated in Table 3. A dial type strain gage with a gage length of ⅚ of the length of the cylinder was used to determine E. Poisson's ratio was determined by means of SR-4 gages.[2] Table 3 also compares the physical properties of the steel reinforcing and concretes used in the Virginia Polytechnic Institute and University of Illinois models.

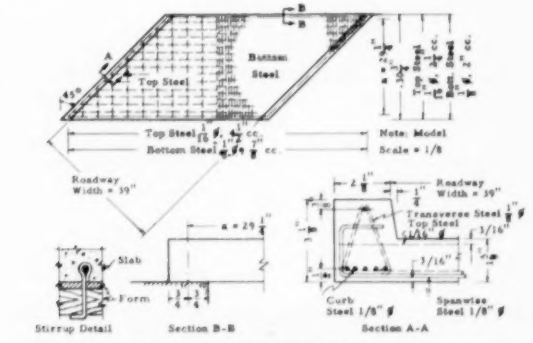

Fig. 12a (top) — Construction details for reinforced concrete model. Fig. 12b (bottom)— Location of strain gages and load points

TABLE 3—COMPARISON BETWEEN VIRGINIA POLYTECHNIC AND ILLINOIS MODELS

Property	Concrete				Reinforcing	
	VPI			Ill.	VPI	Ill.
	2 x 4 in.	4 x 8 in.	6 x 12 in.	6 x 12 in.	⅛ in. ϕ	½ in. ϕ
Poisson's ratio	0.147	0.151				
Modulus of elasticity, psi	3,490,000*	2,690,000*	2,540,000*	3,750,000†	29,050,000	
Ultimate strength, psi	4196	4480	3910	3300	63,900	87,800
Elastic limit, psi					24,800	
Johnson's elastic limit, psi					39,200	
Yield strength @0.001 in. per in., psi					51,000	57,200‡
Elongation, percent					18 in 2 in.	20.4 in 8 in.
Reduction in area, percent					76	

*E at 1000 psi. †Tangent modulus. ‡Yield point.

The skewed slab was first loaded with a 240-lb concentrated load placed at one of the loading stations indicated in Fig. 12 and readings were taken on the strain gages, and of the deflection at several points along each curb and along the axis E-E. This load was then applied at all other loading stations in turn and the entire process repeated for an 800-lb concentrated load. Loads were applied by an hydraulic jack and measured with a Baldwin load cell as shown in Fig. 14. A total of 35 SR-4 gages were attached to the model but the results of only eight are included in this paper. The results for the remaining gages and of the deflections will be published later in detail. A 3500-lb load (equivalent to 219 lb on the VPI model) was used for this test on the Illinois model. Both model and prototype remained uncracked during this loading and both behaved elastically.

The influence lines for strain along axes EE, FF and GG (or $G'G'$) are compared in Fig. 15 for the VPI and Illinois models. These are plotted as

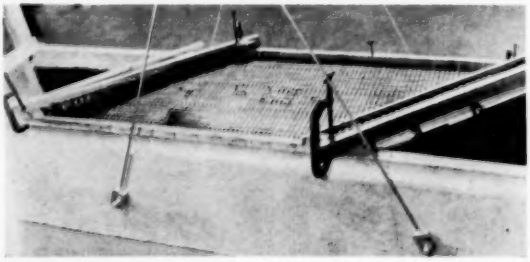

Fig. 13—Top and bottom reinforcement and attached strain gages for reinforced concrete model

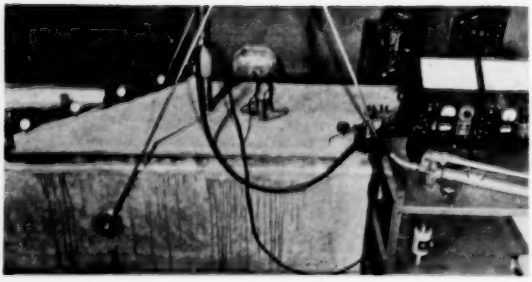

Fig. 14—Test arrangement for the elastic loading of the concrete model slab

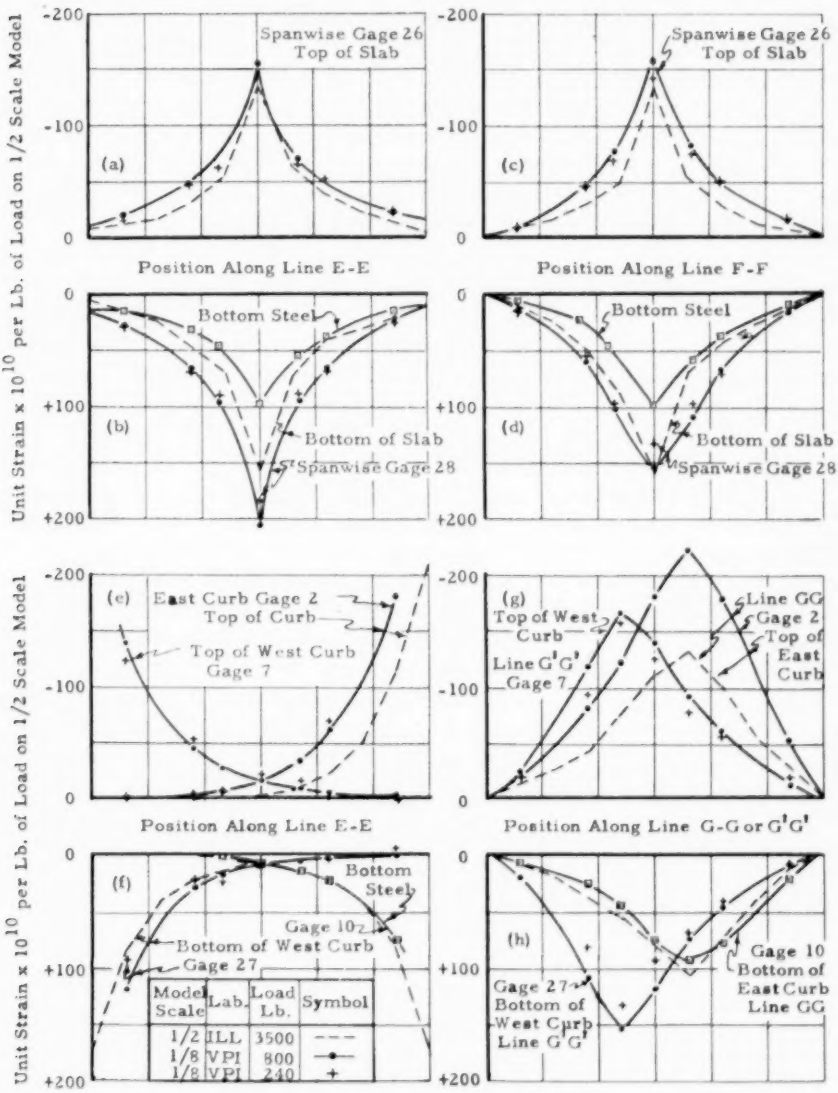

Fig. 15—Comparison between influence lines of Virginia Polytechnic and Illinois models for strain along axes *EE*, *FF*, and *GG* (or *G'G'*)

unit strains $\times 10^{10}$ per lb of load on the ½-scale model. Hence, the strains on the ⅛-scale had to be divided by the load and by the model load ratio of 16 for comparable results. The data in Fig. 15a, b, c, and d for the slab indicates that the two slabs had virtually identical behavior although the smaller model did exhibit slightly higher strains. This difference could be

attributed to the slightly lower modulus of elasticity of the concrete in this small model.

Similar comparisons of strains at the center of the curbs for the two models showed considerably greater divergence (Fig. 15e, f, g, and h) with the small model again being strained to larger amounts. The differences here are probably due to the fact that some honeycombing did develop in the west curb and an even larger amount in the east curb. The east curb exhibited greater strains, as the curves of Fig. 15e and 15g indicate.

After the elastic test with the single concentrated load had been completed, the slab was loaded with one concentrated load at the center of the slab and another equal load 9 in. away at station 9 as indicated in Fig. 16c. This axle load was then increased in increments until the first crack appeared just under the center at a total load of 3250 lb. At that load the crack was only 0.05 mm wide and might possibly have occurred at a slightly smaller load. The Illinois model exhibited a first crack for a similarly placed load of 40,000 lb. This is equivalent to 2500 lb on the VPI model. However, if the tensile strengths were proportional to the compressive strengths for the concretes used for model and prototype, the VPI model load would have been 2500 (4196)/3300 = 3180 lb which is quite close to that of 3250 lb observed. This axle load of 3250 lb was then placed at each of the other 15 stations in turn (Fig. 16c). The crack pattern which developed on the bottom of the slab is shown dotted in Fig. 16b. Practically all cracks paralleled the transverse reinforcing and coincided exactly with it.

The double axle load shown in Fig. 16a was then placed on the slab and the load increased to the ultimate. This occurred at 11,450 lb. The loading

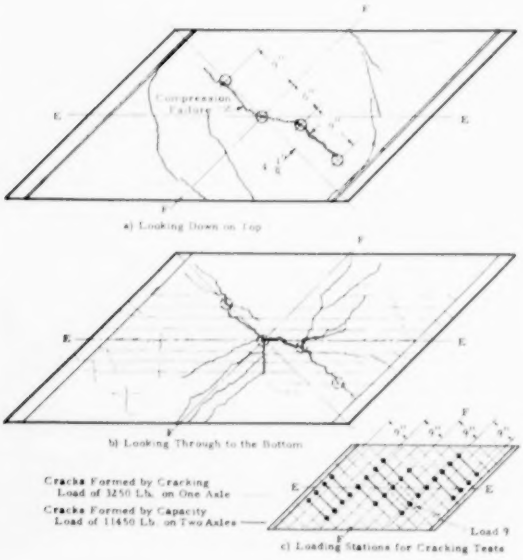

Fig. 16—Cracking test loading and cracking patterns

to the ultimate differed slightly from the Illinois tests for the distributed dead load of 163 psf on the slab and 241 lb per ft on the curbs, necessary to satisfy similitude requirements, were partially omitted from the VPI tests since the dead weight of this model was not augmented. This additional weight would have amounted to 142.8 psf on the model slab and 52.7 lb per ft along the model curbs with a total of 2060 lb. Assuming about one-half of this as effective as a double axle load, the true live load would be about 11,450 − 1030 = 10,420 lb. The ultimate for the Illinois model was dead load plus 155,900 lb. Taking $\frac{1}{16}$ of the live load of 155,900 gives 9750 lb which is within 6.9 percent of the 10,420 lb as corrected.

A further check on the similarity of the model behavior may be made by comparing the final crack patterns of the two slabs. The crack pattern for the VPI model is shown in Fig. 16a and 16b, and parallels closely that of the Illinois model. The latter had a few more tensile cracks in the top of the slab, due no doubt to the distributed load. At the ultimate load on the VPI model, the acute corners had raised off the support by $\frac{1}{2}$ and $\frac{1}{4}$ in.

SUMMARY

No attempt was made in this report to compare the results with any analytical analysis. The data presented for the aluminum model give a detailed account of the variation of stress and deflection at critical points, and should enable anyone so inclined to compare these results with a theoretical solution of the governing differential equation when it is obtained.

The comparison of results for the reinforced concrete models leads the authors to conclude that they will follow the laws of model analysis quite faithfully within the elastic range if suitable techniques in fabrication are followed. It appears that the physical properties of the model concrete will have to be obtained from model cylinders scaled down proportionately. The over-all results are, however, most encouraging and indicate that tests can be made on models of entire structures, rather than on only their component parts, in small laboratories with a minimum of equipment.

ACKNOWLEDGMENTS

The authors wish to express their gratitude for the financial assistance obtained from the Research Corporation of New York, from the Reinforced Concrete Research Council, and from the Virginia Polytechnic Institute Engineering Experiment Station in furthering this research. Thanks are also due the Aluminum Company of America for fabricating the aluminum model and the American Steel and Wire Co. for supplying the $\frac{1}{8}$-in. deformed wire. Acknowledgment is also made of the services of Richard N. Southworth and Newton Elmore who served as research fellows in applied mechanics and of the work of Michael C. Cusano and John R. Ruff who developed parts of this work in conjunction with their MS theses.

REFERENCES

1. Hewson, T. A., "A Nomographic Solution to the Strain Rosette Equations," *Proceedings*, Society for Experimental Stress Analysis, V. 4, No. 1, pp. 9-35.

2. Pletta, D. H., and Frederick, D., "Experimental Analysis," *Proceedings*, ASCE, V. 79, Separate No. 224.

3. Gossard, M. L., Siess, C. P., Newmark, N. M., and Goodman, L. E., "Studies of Highway Skew Slab-Bridges with Curbs, Part II: Laboratory Research," *Bulletin Series* No. 386, University of Illinois Engineering Experiment Station, Feb. 1950.

NOTATION

$\epsilon_x, \epsilon_y, \epsilon_z$ = Unit strain

$\sigma_x, \sigma_y, \sigma_z$ = Unit stress

σ_{xi}, σ_{xo} = Unit stress parallel to the x axis on the inside and outside surfaces of the model, respectively

E = Modulus of elasticity

ν = Poisson's ratio

N = Total thrust

V = Total shear

t = Thickness of slab or abutment

M_{yD}, M_{yA} = Total moment about the y axes along BB in the deck and along AA in the abutment, respectively.

K, K' = Stress concentration factors in the abutment and deck near the fillet, respectively

σ_{xn} = $(\sigma_{xi} + \sigma_{xo})/2$ = Normal stress due to thrust in the abutment

σ_{xm} = $(\sigma_{xi} - \sigma_{xo})/2$ = Bending stress due to moment in the abutment

P = Total applied load

Title No. 51-10

Fundamental Factors in the Drying Shrinkage of Concrete Block*

By GEORGE L. KALOUSEK†

SYNOPSIS

Shrinkage of concrete block dried to stable condition successively at 85, 70, 50, and 25 percent relative humidity was not always related to moisture loss. In one series of tests, block underwent slight expansion for a time while losing moisture at 70 percent relative humidity. Sand and gravel block dried at 25 percent relative humidity showed no expansion during rehumidification at 50 and 70 percent relative humidity although up to 17 percent of the total moisture content was re-adsorbed.

Measurements of apparent surface areas by water vapor and nitrogen adsorption suggested a clue as to the mechanism by which water caused volume changes in concrete block. The amounts of water vapor adsorbed were approximately twice those required to cover the true internal surface as determined by nitrogen adsorption. The amount of this excess water, designated "interlayer" water, was about 0.7 times as large for autoclaved units as for normally cured units. Since the autoclaved block showed a shrinkage about 0.6 times as large as that manifested by the normally cured ones it appeared that the amount of shrinkage may be proportional to the amount of the "interlayer" water. The volume changes with adsorption and desorption of water, at least at moderate to low humidities, could be caused by the change in size of the unit cell of the structure of the cementitious phase. The manner in which the capillary phenomenon may contribute to volume change is considered. The capillary-tension force is probably operative only at high humidities.

INTRODUCTION

Drying shrinkage and attendant cracking of concrete masonry is becoming a serious problem. The need of more information on factors affecting moisture-volume relations in concrete has been receiving consideration during the past few years. In a broad program at the University of Toledo, studies have been carried out on the effects of various physical factors involved in drying shrinkage. Research on the fundamental aspect of the problem with the purpose of explaining the basic causes of shrinkage was a part of the broader program dealing with the shrinkage characteristics of hydrous calcium silicate products in general. This report gives a brief resumé of some of those studies, presents some new data, and advances interpretations of different observations on the phenomenon of drying shrinkage.

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RESUME OF STUDIES PERTAINING TO BASIC FACTORS OF DRYING SHRINKAGE

Unpublished results[1] of length changes and moisture content changes during stepwise drying at successively lower humidities, or rehumidification of dried block at successively higher humidities, could not be satisfactorily explained by any theory on shrinkage. Sand and gravel block underwent small but real expansion during drying at one stage of the cycle and while losing water. Re-expansion of sand and gravel block dried at 25 percent relative humidity (R.H.) was small or negligible at humidities up to 70 percent R.H. although significant amounts of water were adsorbed.

Results of adsorption of nitrogen and water vapor by autoclaved and normally cured block[2] indicated that approximately twice as much water as nitrogen was adsorbed by the units. A large difference in apparent areas is also evident in a comparison of the results of Blaine and Valis[3] obtained by nitrogen adsorption and those of Powers and Brownyard[4] obtained with water. The large differences in water and nitrogen adsorption are considered significant because the diameters of the two molecules (2.7 and 3.1 A) are not significantly different when regarded in terms of the results mentioned. The areas from the two adsorbates should be approximately equal if the adsorption occurs only on surfaces available to these two types of molecules. Jura and Harkins[5] have reported nearly identical areas for anatase (TiO_2) using nitrogen and water as adsorbates. Results on xonotlite, a monocalcium silicate hydrate of large internal surface area, also showed nearly the same areas for water and nitrogen. These results on adsorption raised questions about the manner in which the excess water adsorbed was accommodated in the cementing phase of the block and how it might effect volume changes.

A recently published report[6] on the cementitious solids of autoclaved products calls attention to a series of hydrous calcium silicates having molar comparisons ranging from $4C{:}5S$ to $5C{:}4S$.* These solids, or closely related ones, are the potential cementitious phases of autoclaved concrete block. The mineral tobermorite, $C_4S_5H_5$ representing the low-lime member of the series, has been studied in considerable detail in the research laboratories of the Owens-Illinois Glass Co. It is described in a recent patent[7] under the name of lepisil. Methods of producing this solid in the form of low-density products is given in that patent, which also includes detailed chemistry on the formation of this solid.

Taylor[8] has advanced the hypothesis that the members of the tobermorite series may have a layer structure similar to that of clays and it was suggested by Bernal[9] and also Taylor[10] that the shrinkage phenomenon occurs by the same mechanism in concrete as in clays. X-ray studies have demonstrated that water is adsorbed by clays in highly orientated layers between the layers of alumina and silica. When such water is desorbed by spontaneous drying, the clays shrink and certain lines in the x-ray pattern shift from wider to narrower spacings. Taylor and Bernal observed shifts in lines

*The abbreviated forms of the oxide constituents are used in this report, that is, $C = CaO$; $S = SiO_2$; $A = Al_2O_3$, and $F = Fe_2O_3$.

from 14 to 9 A in the mineral riversideite and also the monocalcium silicate hydrate of the tobermorite series when these were heated at moderately elevated temperatures and 6 mm *Hg* pressure.

X-ray studies on samples of synthetic tobermorite in moist and vacuum dried conditions and also on specimens heated at moderately elevated temperatures were made in the laboratories of the Owens-Illinois Glass Co. Only at about 115 C and higher did the specimens show significant alterations in the patterns. The samples dried at room and slightly elevated temperatures showed minor, but nonreproducible, shifts in some of the lines at lower spacings. Although the x-ray data* are inconclusive on samples dried at room temperature, nonetheless, the possibility that movement of structural water is related to drying shrinkage is indicated. Indirect results on water and nitrogen adsorption by concrete products, and also shrinkage behavior of xonotlite and tobermorite specimens, suggest that the water causing shrinkage is probably not surface adsorbed water. It is assumed that water moving into, or out of, the crystal lattice is one of the underlying causes of volume changes of concrete products. Such water for purposes of discussion is designated as "interlayer" water in the following discussions.

Much of the data considered in this report were obtained on autoclaved and normally cured concrete block, and comparisons are made between the results obtained for the two types of curing. The interpretative statements to be made, although necessarily tentative due to the many factors involved, appeared permissible because of the following considerations. A review[6] of the more recent x-ray data indicate that the lime-rich phases prepared at room temperature by either the reaction of lime, silica, and water or direct hydration of beta C_2S, and the low-lime phases obtained hydrothermally are the same or highly similar in atomic structure. The hydration product of C_3S, as described by Taylor,[8] manifested a slightly different x-ray pattern from the phases just mentioned but was considered to have a structure similar to these. Although the hydrates of beta C_2S and C_3S that form in a cement paste may be altered somewhat by the other constituents present, it is unlikely that the structure would be altered significantly.

In addition to the x-ray data, two different types of experimental data on autoclaved and normally cured block are now available which also indicate that the pattern of behavior of the two products is similar.

Drying shrinkage curves (Fig. 8 and 9) manifest much the same detail for both the autoclaved and normally cured units. The principal and most important difference is that the shrinkage of the autoclaved block is markedly lower than that of the normally cured block. In the report[2] on adsorption tests it is shown that the adsorption isotherms of autoclaved and normally cured products are highly similar. All the preceding considerations therefore suggest that the differences observed in these results for autoclaved and

*Subsequent to the completion of the final version of this report for publication, new x-ray data were obtained which show more conclusively that the structure of a certain type of the hydrous calcium silicate may undergo a marked change during drying in vacuum and rehumidification at ordinary temperatures. The report giving details of the test and the data is pending publication.

TABLE 1—AVERAGE APPARENT SURFACE AREAS PER GRAM OF CEMENT AND RATIOS OF AREAS OBTAINED BY NITROGEN AND WATER VAPOR ADSORPTION ON SIX DIFFERENT COMMERCIAL CONCRETE BLOCK

Test	Adsorbent	Cure	Average of six surface areas (or ratios), sq m per g
Surface area	Water	High-pressure steam	90
Surface area	Nitrogen	High-pressure steam	53
Ratio of areas	Water/nitrogen	High-pressure steam	1.7
Surface area	Water	Steam at atmospheric pressure	83
Surface area	Nitrogen	Steam at atmospheric pressure	31
Ratio of areas	Water/nitrogen	Steam at atmospheric pressure	2.7
Ratio of areas	Water	High-pressure steam/ Steam at atmospheric pressure	1.1
Ratio of areas	Nitrogen	High-pressure steam/ Steam at atmospheric pressure	1.7

normally cured products are ones of degree but not kind. The preceding discussion applies to products properly autoclaved at steam pressures in the range of about 100 to 150 psi. The solids forming at higher pressures (about 200 psi and higher) will differ markedly in x-ray patterns and performance from those autoclaved at the lower pressures mentioned.

INTERPRETATION OF RESULTS OF SURFACE AREAS

Block for the surface area tests were produced commercially from six different aggregates. The methods and details of tests have been published.[11] One-half of each batch was cured normally (steam at atmospheric pressure) and the other half was autoclaved. Only the average results of the complete data[2] calculated in terms of surface area per gram of cement are presented in Table 1. Water adsorption tests were carried out at 15 percent relative humidity to avoid capillary condensation.

The apparent surface areas by water adsorption are approximately the same for both types of curing. The true internal surface area measured with nitrogen was considerably lower than that shown by water adsorption. However, the area available to nitrogen was much larger for the autoclaved than normally cured block. These results permit computation of the amounts of the interlayer water by subtracting the amount of water required to cover the surface available to nitrogen from the total amount adsorbed. The computations,* using the average results in Table 1 which represents a mix yielding 20 block per 94 lb of cement, show that 0.049 lb and 0.068 lb of interlayer water was adsorbed by autoclaved and normally cured block, respectively.

*These computations were based on the assumption, not entirely valid as noted below, that 1 g of cement yielded 1 g of the binder. In the autoclaved block each gram of cement formed approximately 1.25 g of binder of an average C/S ratio of about 1.2 and containing approximately 12 percent fixed water. This yield is readily computed from the known CaO content of the cement and the composition of the reaction product of 0.9 to 1.3 C/S as described in reference 6.

In the normally cured block it was estimated that 1 g of cement formed, on the average, 1 g of the binder. In estimating this yield it was assumed, for stated reasons, that the following conditions were obtained regarding the state of the reaction product. (1) As suggested by x-ray data the cement was hydrated 80 percent. (2) The hydrolytic $Ca(OH)_2$, as shown by differential thermal analysis results, had combined with the siliceous fines in all block except sand and gravel and slag, and thereby augmented the amount of the binder. (3) The product had a C/S ratio of about 1.8, this being an average value based on results of different studies. (4) The binder contained 18 percent fixed water.

If these assumptions were correct, or the errors compensated each other, the ratio of the interlayer water in the autoclaved block compared to the normally cured block would be 0.5, which is nearly the same as the ratio of shrinkages. Although this agreement is probably fortuitous, it, nonetheless, suggests that shrinkage may be proportional to the amount of the interlayer water.

TABLE 2—TOTAL SHRINKAGE OF CONCRETE BLOCK AT 25 PERCENT R.H. FROM SATURATED CONDITION, AND THE PORTION OF SHRINKAGE THAT OCCURRED AT 70 PERCENT R.H. BEFORE BLOCK WERE MOVED TO 25 PERCENT R.H.*

Aggregate	High pressure cure		Normal cure	
	Total shrinkage, percent	Shrinkage at 70 percent R.H., percent of total	Total shrinkage, percent	Shrinkage at 70 percent R.H., percent of total
Sand and gravel	0.013	46	0.030	62
Cinder	0.025	60	0.046	56
Expanded shale	0.025	68	0.041	59
Expanded slag	0.029	59	0.050	50
Sintered shale	0.034	64	0.056	54
Pumice	0.045	59	0.072	61
Average of six	0.029	59	0.049	57

*All results referred to lengths of saturated block.

The autoclaved block, therefore, adsorbed about 0.7 times as much interlayer water as did the normally cured units. The results of shrinkage at 25 percent R.H. of block stabilized at 70 percent R.H. in Table 2 show that the autoclaved units shrink only about 0.6 times as much as did those cured normally. The fact that desorption measurements were made at low humidity and the shrinkage measurements at higher humidities may have some effect on the comparison made. However, this comparison is made in terms of ratios, not actual quantities, and it seems that the ratios of the actual shrinkages at low humidities at which the adsorption tests were made would show a value not too different from that observed. Such an assumption may be made because both the autoclaved and normally cured specimens showed highly similar adsorption isotherms suggesting a similarity in pore distribution over the humidity range of 15 to 93 percent R.H.

Although the ratios of interlayer water (0.7) and shrinkages (0.6) agree reasonably well, no definite conclusions can be drawn regarding the significance of these results. Attention is merely directed to this agreement because it suggests the possibility that the amount of interlayer water may be proportional to shrinkage. Further experimentation on these ratios appears highly desirable.

FUNDAMENTALS OF THE CAPILLARY PHENOMENON

Capillary tension theory

Drying shrinkage has often been attributed to the gel-like properties of hydrated cements and considered to be essentially a capillary phenomenon. According to this theory the stress producing a volume decrease is the surface tension of water in partially filled capillaries.

For purposes of illustrating the manner in which capillary forces may be operative, a highly simplified diagram of spheres of hydrated cement paste is presented in Fig. 1. The sphere at the left illustrates a saturated paste and, therefore, one in an expanded condition. The second sphere represents a partially dried paste with the water, as shown by the cut-away section, partially receded into the capillaries. The development of the meniscus results in the development of a stress which tends to compress the mass.

Fig. 1 — Diagrammatic sketch illustrating the manner in which the capillary-tension stress may effect volume change of hydrated cement

Since the cement gel is an elastic or nearly completely elastic solid, the stress so developed decreases the volume. It is known, with exceptions to be presented, that as concrete is dried at decreasing humidities the cement gel continues to shrink. The third sphere in Fig. 1 represents a completely dried paste with the capillaries completely free of liquid water. The paste should have re-expanded if only the capillary tension stress was involved. Apparently other forces are operating during drying shrinkage.

Powers and Brownyard[12] previously discussed the limitations of the capillary phenomenon and advanced two hypotheses to explain shrinkage. They suggested that desorption of the first layer of adsorbed water could produce shrinkage but, "that it remained for future experiments to prove or disprove the deduction that the part (of water) responsible for volume change is the adsorbed water." The present data do not support the hypothesis because the major portion of shrinkage of concrete occurs at vapor pressures definitely above those at which the first layer of water is desorbed. The second hypothesis is based on the premise that potential swelling pressure of the gel equals the capillary tension force, it being assumed that shrinkage was entirely independent of the amount of water in the capillaries.

This highly valuable contribution presents the mathematical equations involving two opposing stresses. The derivations by Powers and Brownyard were made for gels considered as consisting of touching spheres, 140 A in diameter, and containing pores 20 to 40 A in diameter. The interstitial spaces between spheres are called capillaries. Undoubtedly the basic equations used by Powers and Brownyard would be applicable for treating the present data. In the present report, however, attention is being directed to shrinkages occurring at high humidities and, therefore, the capillary-tension force is regarded as possibly contributing to shrinkage.

Recently Woods[13] reported that the water in capillaries of hardened cement paste begins to evaporate completely at about 80 percent R.H. Therefore, even at this high humidity the capillaries are partially filled and capillary tension stress should develop. Also at this high relative humidity the water in the pores of the gel (20 to 40 A in diameter) could not evaporate and, therefore, the shrinkages observed at high humidities are probably not due to this cause. It is assumed that capillary-tension force produces the shrinkage at high humidities, and results of some direct tests will be presented to elucidate further this hypothesis.

The stress attributed to the desorption of interlayer water can be treated

Fig. 2—Electron photomicrograph of synthetic tobermorite ($C_4S_5H_5$)

similarily as that caused by desorption of gel water as presented by Powers and Brownyard.

Potential solids of autoclaved concrete products

Fig. 2 is an electron photomicrograph of synthetic tobermorite. The plate-like crystals are extremely thin and the surface area of this solid available to nitrogen was 50 sq m per g. Fig. 3 shows the crystalline structure of the cementitious phase of an autoclaved lime-quartz paste prepared with a water to solids ratio of 0.7. This photograph gives some idea of the pore structure of the cementitious phase of the autoclaved product. This solid showed an x-ray pattern nearly identical to that of tobermorite and therefore it is assumed that tobermorite of composition of about *CSH* is the cementing solid. The surface area available to nitrogen was 43 sq m per g and the area computed from water adsorption data was 90 sq m per g.

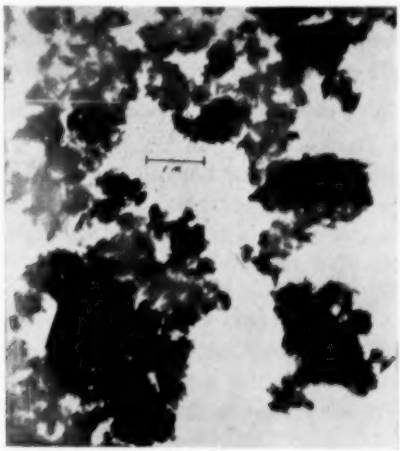

Fig. 3—Electron photomicrograph of *CSH* of the tobermorite series made of lime and quartz using a water to solids ratio of 0.7

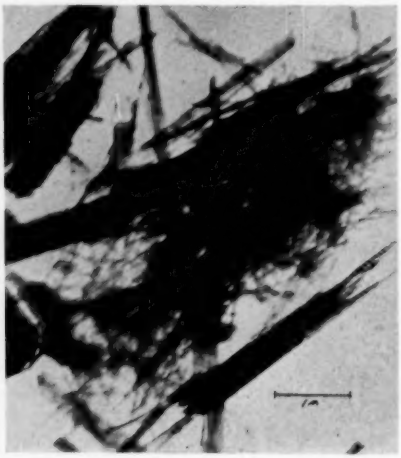

Fig. 4—Electron photomicrograph of synthetic xonotlite (C_6S_6H)

Fig. 4 shows the lath-like crystals of xonotlite. Surface areas of 20 to 30 sq m per g were only slightly higher (about 10 to 25 percent) for water than nitrogen adsorption. Considering the magnitude of difference of approximately 200 percent for the tobermorite and concrete block, and the size of the errors involved in the measurements it may be said that xonotlite showed the same surface area for both adsorbates. The observation is important for three reasons. First it may be assumed that the measurements of areas by water vapor adsorption at 15 percent R.H. are valid results. That is, the large amounts of excess water adsorbed by concrete block is not due to capillary condensation or formation of multi-molecular layers. If the theory is correct that shrinkage of concrete products is due to desorption of interlayer water then it may be expected that xonotlite which apparently does not contain interlayer water should manifest no shrinkage when completely dried. Finally, bars of xonotlite should show a shrinkage-swelling curve during continuous drying as required by the capillary tension theory of drying shrinkage because only the capillary tension force would be operative.

Drying shrinkage of xonotlite and tobermorite specimens

Bars of xonotlite, 1 x 1 x 8 in. and of 18 lb per cu ft density, were made according to methods described previously.[14] These were saturated in water for 3 days and then exposed in the laboratory air to dry. The results of length changes and moisture contents obtained by methods used in testing concrete block[11] are given in Fig. 5. As required by the capillary tension theory the bars shrank while losing water but only down to a certain moisture content; in this test it was 15 percent. The bars then re-expanded while losing further moisture because the forces producing the strain started to decrease as the capillaries began to go dry. The specimens, however, did not re-expand to the original length during drying in the laboratory air. Additional drying for three days in vacuum at room temperature produced

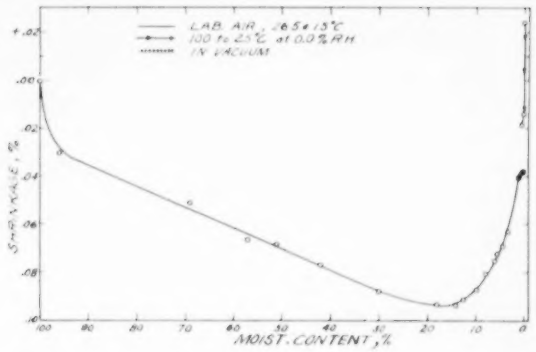

Fig. 5—Relation between moisture content and shrinkage of xonotlite specimens of 18 lb per cu ft density

only a small further re-expansion. It seems that xonotlite undergoes a deformation of approximately 0.03 to 0.04 percent when dried at room temperature. The bars from these tests and other specimens, when resaturated, regained the original saturated length. Drying of these bars at temperatures from 100 to 250 C showed (after cooling) the very small deformation between + 0.005 to − 0.017 percent. Therefore the small drying shrinkage at room temperature may be due to some type of mechanical strain arising from the capillary tension stress. Although the bars underwent a slight shrinkage it is relatively small compared to the shrinkage of the tobermorite bars. The results, no matter what the cause of the shrinkage, seem to show that capillary tension force produces a strain which is released when the capillaries are emptied of all liquid water.

The same tests as described in the preceding paragraph were made on the monocalcium silicate hydrate (*CSH*) of the tobermorite series. Results are given in Fig. 6. In the general features, up to the final stages of drying, this curve agrees with that of xonotlite. Shrinkage occurred and was followed by re-expansion down to the remaining 5 percent moisture content (this

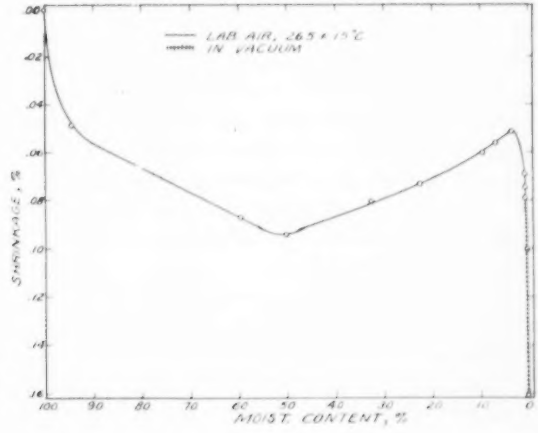

Fig. 6—Relation between moisture content and shrinkage of tobermorite specimens of 18 lb per cu ft density

corresponds to 5 percent water by weight of the solid), then followed a large and abrupt shrinkage. The bars were also dried in vacuum for three days. The large shrinkage, 0.16 percent, is assumed to be caused by the spontaneous desorption of the interlayer water. The drying shrinkage at 100 C was 0.24 percent.

In other tests of tobermorite specimens it has been observed that the re-expansion, immediately following the lowest value of shrinkage, would vary. The lowest value observed for re-expansion was 0.02 percent, and this was on bars dried slowly. This is ascribed to development of the two opposing stresses to varying degrees. If these forces would be exactly equal, the portion of the curve between the lowest shrinkage due to capillary tension to start of the abrupt shrinkage would be a horizontal line.

EFFECTS OF VARIATIONS IN CEMENT COMPOSITION ON SHRINKAGE

The potential C_3A effects the drying shrinkage to a much greater degree than do the other compounds in portland cement. It has been shown by Gonnerman[15] and others that the drying shrinkage increases with increases in potential C_3A. It is not yet clear if the same relation would be obtained for cements in which the potential and actual amounts of C_3A were equal.

The relation between potential C_3A and drying shrinkage of mortar bars cured 7 days and 3 months and dried for 1 year is shown by a plot of the Gonnerman data in Fig. 7. The results for specimens cured 7 days show a better degree of correlation than those of specimens cured 3 months. The same shrinkage data are also plotted against the A-F ratios by weight. In this plot the correlation of the specimens cured 3 months is better than that with the C_3A, but the 7-day specimens show a larger spread using the A-F ratio than shown with the C_3A. The spread is relatively large in both methods of plotting and this probably should be expected. As indicated by the trends both A and F are factors in drying shrinkage. Since different cements of a given potential C_3A content or of a given A-F ratio will undoubtedly contain different amounts of actual C_3A, the dark prismatic phase, C_4AF and related solid solution, and glasses of various compositions it would not be surprising if these variations affect shrinkage.

The nature of the products in which the A and F are present in the hydrated cement is still unknown. One can only speculate on reasons why variations in the A-F ratios have such marked effect on the properties of concrete. The most promising working hypothesis regarding the role of A and F is the simplest one; that is, the A and F to some extent become constituents of the crystal lattice in the silica layer of the hydrous calcium silicate.

Lerch[16] has shown that alkalies, and also gypsum in case of certain cements, affect the drying shrinkage. These are two variables not considered in the Gonnerman data. For this reason it could be expected that the spread in data in Fig. 7 might be altered somewhat if these two variables were included. Although the alkalies may have contributed to shrinkage in the past use of

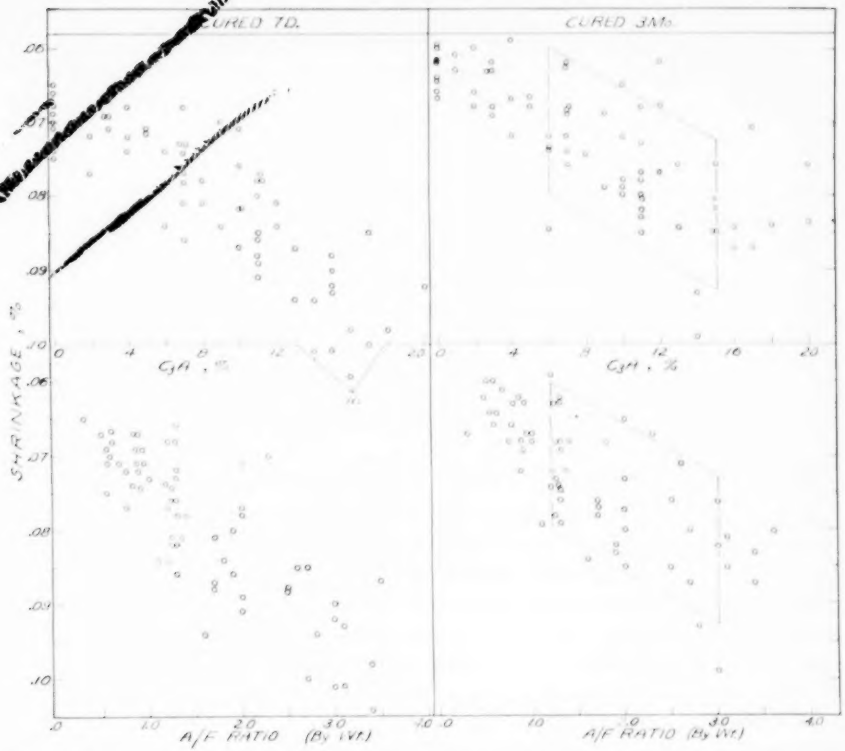

Fig. 7—Plot of Gonnerman[1] data showing relation between 1-year shrinkage and C_3A content and A-F ratio

concrete, this probably will not occur again to as great extent since the producers have taken measures to reduce the alkali content of the cements. Giving consideration to the composition of cements, the following analysis is advanced regarding the effects of cement on drying shrinkage.

Probably all the cements used in concrete block production vary in potential C_3A content between about 6 and 15 percent and A-F ratios between 1.25 and 3.0. The cements of lower A-F ratios are special cements and also give significantly lower strengths than do the others. Therefore, these cements, although showing favorable shrinkage results, will not be used much. Examination of the data within the parallelograms, arbitrarily drawn as including cements generally used in block production, show the possible ranges in shrinkage to be anticipated. The lowest shrinkage is approximately 0.06 percent and the highest 0.09 percent, and it would appear that cement composition would affect appreciably the drying shrinkage. However, these are the extreme limits (excluding the old high-alkali cements) in range of shrinkages and it may be assumed that a high portion of concrete block are made with cements showing a smaller spread in shrinkages than suggested by the values of 0.06 to 0.09 percent.

TENTATIVE THEORY OF DRYING SHRINKAGE

It is inferred, from the data presented for specimens of xonotlite and tobermorite, that two stresses develop during the drying of saturated specimens. One stress is due to capillary tension effect and is developed at humidities at which the pores in the cementing solid are only partially filled with water. This stress is released when the water is completely evaporated from the capillaries. Because the solids possess a marked degree of elasticity the release of the compressive stress results in a re-expansion.

The other stress ascribed to the spontaneous desorption of interlayer water is productive only of shrinkage during drying. This stress develops simultaneously at the higher humidities with that of elastic re-expansion caused by the complete evaporation of water from the capillaries. These two stresses being of opposite signs will tend to balance each other and diminish volume change at certain humidities.

INTERPRETATION OF SHRINKAGE-MOISTURE CHANGE OF CONCRETE BLOCK

Results[1] of shrinkage-moisture loss in concrete dried to equilibrium successively at humidities of 85, 70, 50, and 25 percent R.H. are presented in Fig. 8. Normal drying at 85 percent R.H. resulted in a relatively substantial amount of shrinkage for each decrement of moisture but the curve flattened as equilibrium was approached. At this humidity, the shrinkage is probably caused largely or entirely by capillary-tension stress, that is, the capillaries are only partially filled. This is in accordance with the observation reported by Woods.[13] Although the volume was stabilized at 85 percent R.H., water continued to be lost from the specimen. This represents a lowering of the water level in the capillaries. Even after the specimens were transferred to

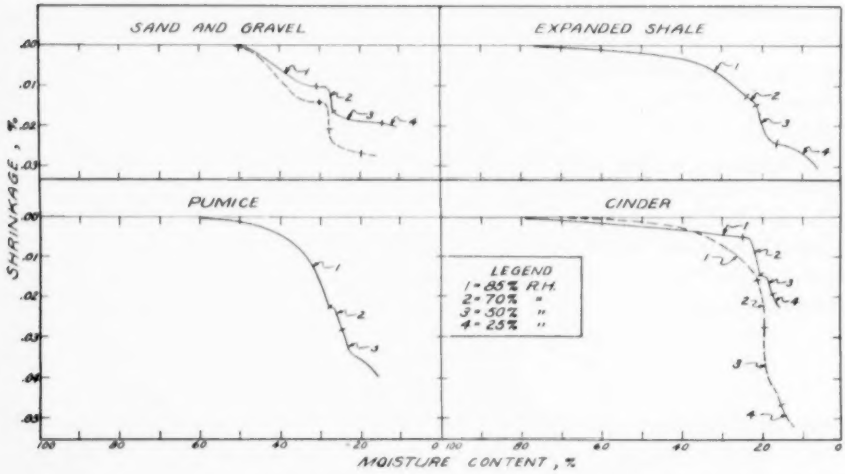

Fig. 8—Shrinkage-moisture content relation of autoclaved and normally cured block dried at different humidities as indicated

70 percent R.H. storage, the volume still remained nearly constant for a period during further loss of water. Although some of the larger capillaries were emptied at this humidity and caused some elastic expansion, it is possible that some interlayer water was being also desorbed. The simultaneous development of the two stresses, of opposite signs but of nearly equal magnitude, could explain the constancy of volume. The elastic re-expansion in the first set of sand and gravel block tested was greater than the shrinkage when these were transferred from 85 percent to 70 percent R.H. The apparent result was an expansion of the units during drying. This amounted to only 0.004 percent but had to be considered real because a check of the temperature and humidity records revealed that both conditions were constant throughout the test.

The relatively large shrinkage with only small loss of water, following the flat portion of the curve, represents the full or nearly full development of the stress due to desorption of structural water. The opposing re-expansion stress is relatively small compared to the shrinkage stress. It may be noted that the rapid shrinkage extended somewhat into the 50 percent R.H. segment of the curve but the cause of this is not clear. Shrinkage stress developing at humidities of about 25 percent R.H. and lower may be assumed to be due largely to desorption of interlayer water; however, surface adsorbed water is also desorbed.

The other curves in Fig. 8 show somewhat the same general effect as that exhibited by the sand and gravel block. The significant observation is the generally sharp shrinkage at about 70 to 50 percent R.H. for small water loss.

It must be emphasized that the curve in Fig. 8 for sand and gravel block is rather exceptional in indicating the apparent balance of two stresses starting at 85 percent R.H. and continuing into the 70 percent R.H. portion of the drying cycle. Generally this inflection in the curve is not so pronounced and in many drying tests at 70 percent R.H. was not observed at all. Some block instead of manifesting a negligible shrinkage with relatively large loss of water at 70 percent R.H. actually showed a relatively large shrinkage with only small loss in water. Drying shrinkage-moisture content results illustrating the different types of drying shrinkage curves at 70 percent R.H. are given in a previous publication (Fig. 8 and 9 of reference 11).

Results observed during stepwise re-humidification of sand and gravel block dried at 25 percent R.H. are shown in Fig. 9. Re-humidification, first at 50 percent R.H. and then 70 percent R.H., resulted in no significant increase in volume. Moisture, however, was being adsorbed continuously throughout most of each of the 30-day periods as shown by the curves in the lower part of Fig. 9. Other block, data for which are given in another report,[1] did not show this constancy in volume, but generally the expansion going from 25 to 50 percent R.H. was small.

Because the volume changes during re-humidification at 50 percent R.H. were nil or small, while water was being absorbed, it may be assumed that two stresses of nearly equal magnitudes but opposite signs were developed. These

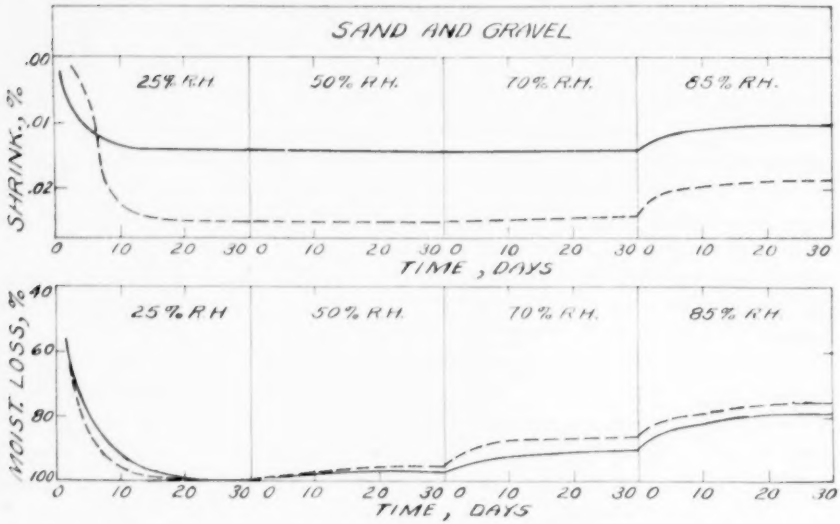

Fig. 9—Relation between volume and shrinkage of dried sand and gravel block during re-humidification at different humidities

stresses tending to expand the solid may be attributed to adsorption of interlayer water. The manner in which the shrinkage stress may develop is not yet clear. If the capillaries are partially filled at the stated humidity it may be assumed that the stress is due to capillary tension effect. If, on the other hand, the water is present as lenses around the points of contact between the particles of cementitious phase as described by Powers and Brownyard,[12] then it would appear that the mechanism advanced by these authors would explain the constancy of volume noted. An unequivocal explanation of this phenomenon will require more complete knowledge on the structure of the cementitious phases.

At 70 percent R.H. it may be assumed that the capillaries become partially filled and that a compressive stress due to the capillary tension is developed. At 85 percent R.H. it may be that the capillary tension stress is developed to a marked degree and this could be the reason why previously dried concrete block do not expand very much at high humidities. From the standpoint of concrete block technology, this behavior of concrete is important because block dried for the purpose of pre-shrinking them can be stored even at high humidities without undergoing marked re-expansion.

SUMMARY

1. Results of water and nitrogen adsorption indicated that approximately twice as much water as nitrogen expressed in terms of surface areas was adsorbed by the monocalcium silicate hydrate of the tobermorite series, and also by autoclaved and normally cured concrete block. Xonotlite, also a

hydrous monocalcium silicate, which undergoes comparatively small drying shrinkage, on the other hand, showed nearly the same surface area for both adsorbates. This and other considerations were interpreted as indicating that the water producing drying shrinkage may be accommodated in the atomic structure and was designated as interlayer water.

2. Tests on specimens of xonotlite and tobermorite showed that the capillary tension stress was productive of shrinkage during the earlier stages of drying up to a certain moisture content and then the specimens re-expanded. At the end of complete drying the re-expanded length of the xonotlite bars remained constant. The partially re-expanded tobermorite bars considered to contain interlayer water showed a large shrinkage starting at a moisture content of approximately 5 percent. This shrinkage is considered as being caused by the spontaneous desorption of interlayer water.

3. Concrete block undergo shrinkage at high humidities which may be caused largely by the capillary-tension stress. Since the elastic strain is released, concrete should re-expand at some lower humidity. This was observed only in one test. It appears possible that simultaneously with the elastic re-expansion another stress due to the desorption of interlayer water is developed. These two stresses, being of opposite signs tend to balance each other at certain high humidities.

4. Autoclaved and normally cured concrete block made of a given aggregate show highly similar volume-moisture change curves and adsorption isotherms. The autoclave specimens, however, undergo less shrinkage. A similarity in the behavior of the products cured by the two different methods is to be expected because x-ray data show that the cementitious solids are highly similar structure-wise. The principal difference between the two is in the composition of the hydrates.

ACKNOWLEDGMENT

The author is deeply indebted to Dr. F. M. Lea for constructive criticisms of the original draft of the report and for highly valuable suggestions relating to some of the experimental tests.

REFERENCES

1. Selden, J. K., Kalousek, G. L., Rohr, W. G., and Brewer, S., "Volume Change Characteristics of Block and Wallettes," *Final Report on Wallettes* No. 1-78, Office of the Chief of Engineers, Department of the Army, Washington, D. C.

2. Gleysteen, L. F., and Kalousek, G. L., "Simplified Method for the Determination of Surface Area of Concrete Products," *Final Housing Research Report*, Housing and Home Finance Agency, Office of the Administrator, Division of Housing Research, Washington, D. C.

3. Blaine, R. L., and Valis, H. J., Research Paper No. 1967, *Journal of Research* (National Bureau of Standards), V. 42, 1949, pp. 257-267.

4. Powers, T. C., and Brownyard, T. L., "Studies of the Physical Properties of Hardened Portland Cement Paste. Part 3—Theoretical Interpretation of Adsorption Data," ACI JOURNAL, Dec. 1946, *Proc.* V. 43, pp. 469-504.

5. Jura, G., and Harkins, W. D., *Journal of American Chemical Society*, V. 66, 1944, p. 1357.

6. Kalousek, G. L., "Studies on the Cementitious Phases of Autoclaved Concrete Products Made of Different Raw Materials," ACI JOURNAL, Jan. 1954, *Proc.* V. 50, pp. 365-380.

7. U. S. Patent No. 2,665,996 issued Jan. 12, 1954, "Hydrous Calcium Silicates and Method of Preparation," G. L. Kalousek assignor to Owens-Illinois Glass Co., Toledo, Ohio.

8. Taylor, H. F. W., *Journal of the Chemical Society*, V. 726, Dec. 1950, pp. 3682-3690.

9. Bernal, J. D., "The Structures of Cement Hydration Compounds," Third International Symposium on the Chemistry of Cement, 1952, Cement and Concrete Assn., London, England.

10. Taylor, H. F. W., *Journal of the Chemical Society*, V. 733, Jan. 1953, pp. 163-171.

11. Kalousek, G. L., O'Heir, R. J., Ziems, K. L., and Saxer, E. L., "Relation of Shrinkage to Moisture Content in Concrete Block," ACI JOURNAL, Nov. 1954, *Proc.* V. 50, pp. 225-240.

12. Powers, T. C., and Brownyard, T. L., "Studies of the Physical Properties of Hardened Portland Cement Paste. Part 4—The Thermodynamics of Adsorption," ACI JOURNAL, Jan. 1947, *Proc.* V. 43, pp. 549-604.

13. Woods, Hubert, *Rock Products*, V. 56, Oct. 1953, p. 202.

14. U. S. Patent No. 2,547,127 issued Apr. 3, 1951, "Calcium Silicate of Microcrystalline Lath-Like Structure," G. L. Kalousek assignor to Owens-Illinois Glass Co., Toledo, Ohio.

15. Gonnerman, H. F., *Proceedings*, American Society for Testing Materials, V. 34, Part II, 1934.

16. Lerch, W., *Proceedings*, American Society for Testing Materials, V. 46, 1946.

Title No. 51-11

Test Hammer Provides New Method of Evaluating Hardened Concrete*

By GORDON W. GREENE†

SYNOPSIS

The Schmidt concrete test hammer provides a convenient method of determining the strength of nearly any concrete mass that has a smooth, flat surface. The test is nondestructive. Results of tests made with the instrument are compared with results of compression and flexure tests. Factors causing variation in results with each test method are discussed briefly, and the accuracy of the test hammer is compared with that of the testing machine.

INTRODUCTION

Frequently structural engineers are faced with the difficult problem of measuring the strength of concrete in existing structures. Usually core samples cut from the concrete are relied upon to provide the desired information. However, the expense and delay involved cause this method to be used only as a last resort. The need for a simple and reliable test suitable for use on existing concrete structures is apparent.

In 1948 a Swiss engineer, Ernst Schmidt, developed the idea of a test hammer which could be used to measure the strength of concrete rapidly and inexpensively without damage to the structure. His data were presented to the Swiss Federal Materials Testing and Experimental Institute in Zurich, where the hammer was constructed and extensively tested.[1,2,3]

DESCRIPTION

The test hammer utilizes the principle that the rebound of a steel hammer is proportional to the compressive strength of the concrete. There are two models of the Schmidt concrete test hammer at the present time (Fig. 1). In Model I (Fig. 1a) a pair of springs provides a uniform propelling force for the hammer which moves within a tubular frame. The frame is provided with a base to hold it perpendicular to the surface of the concrete being tested and with a scale to measure the rebound of the hammer.

To use the hammer the index catch is depressed into the slot beneath the index, which is set about ten scale units below the probable rebound number as determined by a trial test. With the hammer held firmly against the

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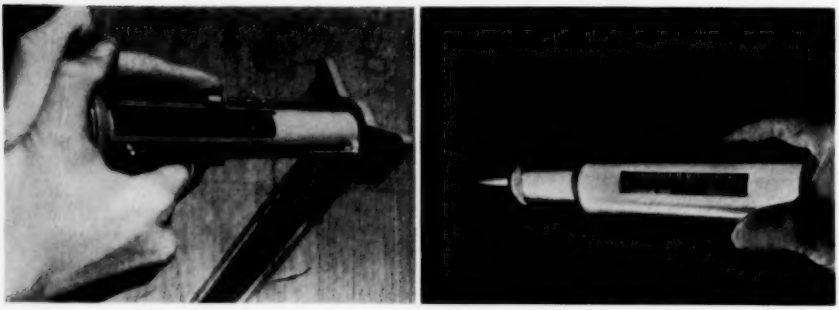

Fig. 1a (left)—Schmidt concrete test hammer, Model I, in use. Fig. 1b (right)—Schmidt concrete test hammer, Model II, in use

concrete to be tested, the trigger is engaged and the hammer is drawn against the springs to the fully extended position. The trigger is then released outwardly. The hammer, as it rebounds, moves the sliding index along the scale. The scale is calibrated to express rebound distance as a percent of the full distance the hammer travels prior to impact.

Several changes have been made in Model II (Fig. 1b). A single, large spring provides the propelling force and the need to reset the index catch and scale after each impact has been eliminated. The chances of error through nonuniform handling of the instrument by an inexperienced operator have been greatly reduced. The hammer is cocked and tripped automatically by pressing the instrument against the surface to be tested allowing the hammer to be operated with one hand if necessary.

DETERMINATION OF CONCRETE COMPRESSIVE STRENGTH

The following instructions for using the Schmidt concrete test hammer are quoted from the booklet supplied with the Model I instrument:

> Testing should be conducted on surfaces that are smooth and uniform. Avoid rough spots, honeycombs, or porous areas. For concrete sections less than 4 in. thick, the rebound may be affected by elastic deformations, and should be backed up by placing a heavy object against the back side.... If the concrete is rough, use the carborundum stone (supplied with the instrument) to prepare a smooth, flat surface 5 or 6 in. square.
>
> Take readings with the concrete test hammer at six to ten points on the selected area. For each impact of the hammer, move the apparatus about an inch to avoid more than one impact at a given point. Determine the average value of R, the rebound number. Two-thirds of the readings should not deviate more than three scale units from the average. If the spread of readings is excessive, repeat the test at a neighboring area to get consistent data.
>
> The compressive strength of the concrete can be found from the relationship between the rebound number R and the strength in psi (given by the curve in Fig. 2). By carefully following the instructions above, the compressive strength of the concrete will be found to lie between the upper and lower standard deviation curves....
>
> The apparatus has been calibrated for use in the horizontal position against vertical concrete surfaces, but it can also be used in any other position by applying a small correction factor....

A chart showing the correction factors for various angles and rebound numbers is included in the instruction booklet.

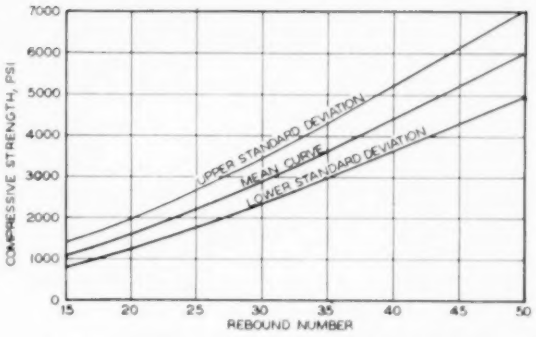

Fig. 2—Relationship between rebound number and compressive strength

The relationship between the rebound number and the compressive strength expressed in Fig. 2 was prepared by Arthur R. Anderson, consulting engineer, Tacoma, Wash., from the tests made by the Swiss Federal Materials Testing and Experimental Institute. Approximately 550 concrete specimens were tested with the hammer to obtain the data presented. The compressive strength of each, measured in a testing machine, was recorded with the mean value of R as determined by ten impacts of the test hammer. In Europe the standard practice is to test concrete in cubic specimens rather than in cylindrical specimens as in this country. To compensate for the higher results obtained from cubes Mr. Anderson reduced the value of each test result by 10 percent when he "translated" the data from metric units.[4]

Tests made with the Model II hammer show that R values slightly lower than those expressed with the Model I are obtained. The final results, expressed as compressive strength of the concrete under test, are the same, as the calibration curve supplied with Model II compensates for the difference. The Model I hammer was used in obtaining the data presented in this paper.

CALIBRATION

Unless damaged, the springs will last indefinitely. Check calibration points may be made by placing paper inserts between the hammer and a polished steel surface and measuring the rebound of the hammer.[5] The smooth, flat surface of a large homogeneous rock could also be used to provide check calibrations. The booklet furnished with the Model I test hammer mentions two simple ways to inspect the operation of the instrument. (1) The hammer should start to slide in the frame when the instrument is inclined 17 to 19 deg from the horizontal. (2) The weight of the hammer should cause the index catch to register seven units on the scale when the hammer is held vertically upward.

TEST HAMMER AND COMPRESSION MACHINE COMPARED IN LABORATORY TESTS

Fig. 3 shows graphically the results obtained when ninety-four 6 x 12-in. cylinders were tested with the test hammer prior to their being broken in a compression machine. The tested concrete samples were made with aggregates from three sources. About 60 different concrete mixes were repre-

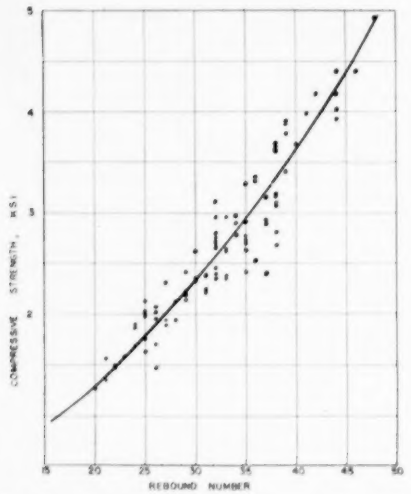

Fig. 3—Comparison of rebound number and the compressive strength as measured by compression machine

sented by the cylinders. Maximum aggregate sizes varied from ½-in. to 1½ in. and cement factors varied from approximately 4 to 7 bags per cu yd. The compression tests were made in three different laboratories over a 1-year period. The rebound number reported represents the average of six impacts on the base of each cylinder, the only smooth, plane surface available for the test.

The results of the tests, as shown in Fig. 3, follow the lower standard deviation curve rather than the mean curve supplied by the manufacturers of the instrument. This difference may be due partially to the fact that the curves supplied with the instrument are based on a greater variety of concrete mixes and conditions of testing. The 10 percent reduction of the cube strengths made by Anderson in preparing the curves is, moreover, very conservative. Gonnerman reports that 6-in. and 8-in. cubes will test 18 and 13 percent higher than 6 x12-in. cylinders.[6]

It is not possible from these few observations to accurately calculate the standard deviation for any given compressive strength. Therefore, to measure and compare the standard deviation of the test hammer and the compression machine, thirty 6 x 12-in. cylinders were prepared from five identical batches of concrete. These cylinders were tested with the hammer and then in a compression machine. The R values obtained by using the test hammer on the cylinders were converted to compressive strength readings by using the curve in Fig. 3. Results are shown in Table 1.

These results indicate that the standard deviation in tests made with the Schmidt concrete test hammer may be larger than that obtained in conventional compression tests but the standard deviation may be less than previously reported[4] (Fig. 2).

The correlation between compressive strength and rebound number shown

TABLE 1—COMPARISON OF RESULTS OBTAINED WITH THE TEST HAMMER AND A COMPRESSION MACHINE

	Test hammer		Compression machine	
Age of specimens, days	7	28	7	28
Number of specimens	12	18	12	18
Average compressive strength, psi	2190	4070	2230	4210
Standard deviation, psi	229	301	142	238
Coefficient of variation, percent	10.5	7.4	6.4	5.7

TABLE 2—RESULTS OF TESTS SHOWING CORRELATION BETWEEN REBOUND NUMBER AND COMPRESSIVE STRENGTH

Age, days	Block from batch A		Block from batch B		Average compressive strength as measured by test hammer, psi
	R	psi	*R*	psi	
1	17.0	980	17.3	1000	990
3	25.4	1780	24.1	1640	1710
5	29.5	2300	28.2	2080	2190
7	31.4	2450	30.8	2380	2420
10	31.5	2470	33.5	2720	2600
14	33.0	2660	34.2	2810	2740
21	34.3	2820	35.7	3030	2930
28	36.4	3130	37.5	3290	3210
40	37.0	3220	38.0	3360	3290
60	40.2	3680	39.4	3560	3620
90	40.6	3740	41.8	3930	3840

Age, days	Cylinders from batch A		Cylinders from batch B		Average compressive strength as measured by machine, psi
	No. cylinders	Average compressive strength, psi	No. cylinders	Average compressive strength, psi	
3	5	1710	2	1830	1740
5	2	2000	2	2080	2040
7	8	2210	2	2380	2250
10	2	2360	2	2700	2530
14	2	2730	2	2770	2750
21	2	2740	2	2890	2810
28	9	3050	3	3140	3070
40	2	3130	2	3230	3180
60	2	3520	2	3630	3570
90	2	3680	2	3810	3740

in Fig. 3 is based on rebound measurements made on 6 x 12-in. cylinders. To test the validity of the curve when the hammer is used to measure widely differing strengths in a larger mass of concrete, a study was made of the increase in compressive strength with age as measured by both methods using appropriate specimens.

Two 24 x 24 x 18-in. blocks, forty-six 6 x 12-in., and eleven 4 x 8-in. cylinders were made from two batches of the same concrete mix. The cylinders were tested in a compression machine at various ages from 3 to 90 days. The blocks were tested with the hammer at the same ages and the *R* values were converted to compressive strengths using the curve in Fig. 3. The test results are shown in Table 2 and Fig. 4.

TEST HAMMER AND COMPRESSION MACHINE COMPARED IN FIELD TESTS

Comparisons of concrete strengths in various structures as determined by the test hammer and the results of compression tests of laboratory cured specimens representing the same concrete showed, at first, rather poor correlation. Evidence indicated that this was due chiefly to careless sampling and improper handling of concrete on the job; therefore, a series of tests was organized to study the problem.

Nineteen 6 x 12-in. and fifteen 4 x 8-in. cylinders were made from four identical batches of concrete delivered to a particular job site. A block of concrete cast in a prismatic 5-gal. can using concrete from one of the four batches was used as a specimen for the test hammer. The cylinders were laboratory cured in the usual manner while the block was handled exactly the same as the concrete placed in the forms. Freezing weather on the fifth

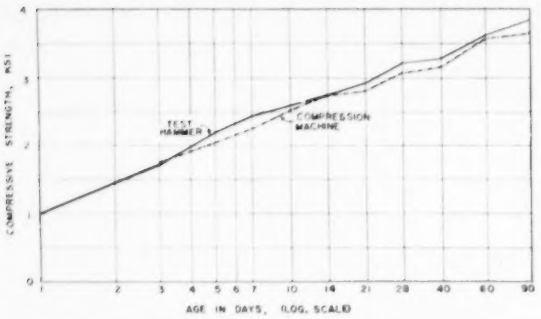

Fig. 4—Comparison of test results using the test hammer and compression machine on different specimens made from the same concrete

and sixth days caused a loss of strength in the block which, of course, was not reflected in the laboratory cured specimens. The cylinders averaged about 500 psi greater compressive strength after the sixth day when tested by either method.

On two occasions compression tests on laboratory cured specimens failed to meet the minimum strengths required. In each case tests made using the hammer on the portion of the structure in question showed strengths far higher than the laboratory tests. In one instance the test hammer indicated an average strength of 3660 psi. The average compressive strength of five cores subsequently cut from the structure was 3520 psi. On the other occasion the hammer showed a strength of 2600 psi. A single core tested at 2540 psi.

TEST HAMMER USED FOR FLEXURAL STRENGTH MEASUREMENTS

Twenty 4-ft x 8-in. x 8-in. concrete beams were tested with the test hammer before being subjected to a flexural test. Each rebound number recorded represented an average of ten impacts in the center one-third of the beam. The rebound numbers were plotted against the modulus of rupture to determine correlation between the two. The results are shown in Fig. 5.

The concrete in the beams contained aggregate up to $2\frac{1}{2}$ in., thus producing rather erratic results in the three-point loading test. On several beams the test hammer yielded higher results when used on the top or bottom surface of the beam as cast. Investigation showed that this was due to large, flat aggregate particles near these surfaces. Concrete specimens made with smaller maximum size aggregates would undoubtedly yield more uniform results with both test methods.

USE OF THE TEST HAMMER WITH LIGHTWEIGHT CONCRETE

Observations made using the test hammer on specimens and structures made of lightweight concrete showed widely differing results. However, for any given type of lightweight aggregate and mixture the rebound numbers proved to be proportional to the compressive strength. Lightweight concrete made with expanded shale aggregates yielded different rebound numbers than concrete made with pumice aggregates at equal compressive strengths. The amount of natural sand used in the mixtures also affected the rebound at any given compressive strength.

It would, therefore, be necessary to experimentally "calibrate" the test hammer for use with any given lightweight concrete mixture. Results of such a calibration are shown below:

Compressive strength f_c', psi	1500	2000	2500	3000
Rebound number, R	30	34	37	41

The concrete tested was a 1:2.25:3.35 mixture by volume using natural sand and 1-in. maximum pumice. Readings made on lightweight concrete show a slightly larger spread than on regular concrete.

FACTORS AFFECTING ACCURACY OF TEST RESULTS

It became apparent during the course of the laboratory and field tests that the smoothness of the surface under test was an important factor affecting the accuracy of test results. Troweled surfaces or surfaces made against metal forms yielded rebound numbers from 10 to 25 percent higher than surfaces made against wood forms. This was especially evident with concrete of high compressive strength. This fact was brought out in a series of tests made on the 24 x 24 x 18-in. blocks previously mentioned. The strengths reflected in tests made on a rough surface reached a maximum below 3000 psi while tests made upon a smooth surface of the same specimen showed strengths in excess of 4500 psi.

When the rough surface was properly prepared by using the carborundum stone the test results were within the limits of accuracy previously reported.

The Swiss Federal Materials Testing and Experimental Institute recommends that the hammer be used only on surfaces where the concrete was covered by forms. Rock or sand pockets will result in very low readings and the presence of air holes, large rock particles, or reinforcing near the surface will cause erratic results. These individual test results should be disregarded.

Concrete mixes made with different rock types show slightly different rebound numbers even though the compressive strengths are the same. This means that the instrument must be calibrated for use with the aggregates encountered in each locality.[3] Experience with widely differing rock types in California indicates this difference is of minor concern.

When successive rebound measurements were made at exactly the same spot a gradual increase in R values was noted. For greatest accuracy the hammer must be moved to a new spot for each test.

Errors in hammer test results caused by imperfect surfaces, variations in

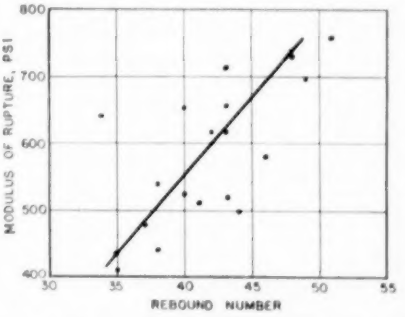

Fig. 5—Comparison of rebound number and the modulus of rupture as measured by the three-point loading test

rock types, and improper use of the instrument are probably no greater than the errors encountered in testing specimens in a compression machine where capping materials and laboratory equipment and techniques are possible sources of error.

Plaster caps are satisfactory when used on normal concrete cylinders but when used with high strength concrete they may result in tests 60 percent below normal. Oiled or dry shot caps may give results up to 38 percent low under the same circumstances.[7] A small error is introduced when the length to diameter ratio of a concrete cylinder is not exactly 2:1.[6,8] If a concrete specimen under test is loaded faster or slower than the rate set forth by standard specifications significant errors are introduced.[9]

Concave or convex caps, undersized caps, bearing block off, inclined axis or ends, and lack of caps all result in errors of varying magnitude.[10,11]

CONCLUSION

The Schmidt concrete test hammer provides a quick, inexpensive method of checking the quality and strength of hardened concrete. It does not offer the consistent accuracy of carefully performed compression tests but its accuracy will probably be sufficient for most work. The instrument is valuable for "trouble shooting" or as a means of checking strength growth in concrete prior to removing forms or shoring. Small differences in the strength of a concrete structure at various points may be detected and measured by the test hammer.

REFERENCES

1. Schmidt, Ernst, "Der Beton-Prufhammer," *Schweiz Bauzeitung*, July 15, 1950.

2. Schmidt, Ernst, "Versuche mit dem neuen Beton-Prufhammer zur Qualitasbestimmung des Betons," *Schweizer Archiv fur angewandte Wissenschaft und Technik*, No. 5, 1951, Druck und Verlang; Buchdruckerei Vogt-Schild AG, Solothurn.

3. Herzig, E., "Versuche mit dem neuen Beton-Prufhammer," *Schweizer Archiv fur angewandte Wissenschaft und Technik*, No. 17, 1951, Druck und Verlang; Buchdruckerei Vogt-Schild AG, Solothurn.

4. Anderson, Arthur R., "A Non-Destructive Concrete Tester," *Concrete*, Aug. 1951, pp. 34-35.

5. Anderson, Arthur R., personal communication dated Nov. 19, 1951.

6. Gonnerman, H. F., "Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete," *Proceedings*, American Society for Testing Materials, V. 25, Part II, 1925, p. 237.

7. Troxell, G. E., "The Effect of Capping Methods and End Conditions before Capping upon the Compressive Strength of Concrete Test Cylinders," *Proceedings*, American Society for Testing Materials, V. 41, 1941, p. 1038.

8. Johnson, James W., "Effect of Height of Test Specimens on Compressive Strength of Concrete," *ASTM Bulletin*, No. 120, Jan. 1943, p. 19.

9. Abrams, Duff A., "Effect of Rate of Application of Load on the Compressive Strength of Concrete," *Proceedings*, American Society for Testing Materials, V. 18, Part II, 1917, p. 364.

10. Gonnerman, H. F., "Effect of End Conditions of Cylinders in Compression Tests of Concrete," *Proceedings*, American Society for Testing Materials, V. 24, Part II, 1924, p. 1036.

11. Kitts, Joseph A., "Report on 2000-lb Concrete to Golden Gate Atlas Materials Company," July 24, 1928.

Title No. 51-12

Ultimate Load Theory and Tests of Cylindrical Long Shell Roofs*

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SYNOPSIS

An ultimate load theory for thin-shell roofs, developed independently by A. L. L. Baker[1] and K. W. Johansen,[2] is presented with modifications as applicable to the design of long, simply-supported, cylindrical shells. The basis of analysis is similar to a rupture theory by H. Lundgren.[3] In addition to design equations for long cylindrical shells, results from exploratory tests are presented in confirmation of the theory. The theory and equations are intended to supplement, not supplant, existing elastic theory.

INTRODUCTION

Although ultimate load (inelastic) methods of analysis for shell roofs are available in foreign literature, there has been little on them in American technical literature. These methods do not replace elastic methods of analysis but do offer certain advantageous simplifications.

In extending the work of previous writers[1,2,3] to a design status, it was found necessary to deal with the following factors in addition to those treated in the reference material: (a) diagonal, or principal, tension stresses resulting from tangential shear; (b) thrust caused by the horizontal components of the difference in tangential shears; and (c) inelastic buckling under longitudinal thrust, and combined transverse thrust and moment at the crown. It is believed that this is the first complete treatment of all factors entering into the ultimate load design of long, simply-supported, reinforced concrete cylindrical shell roofs.

The presentation in this paper is limited to simply-supported, long, thin shells for the purpose of brevity, although the basic principles may be combined with a consideration of elastic deflections to simplify the design of multiple-unit and short shell roofs.

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STRUCTURAL ACTION AT ULTIMATE LOAD

Referring to Fig. 1, the structural action of such shells at ultimate load may be conceived as consisting of the combined resistance of the whole shell acting as a beam with curved cross section and a series of longitudinal strips subjected to slab bending. The effect of the longitudinal strips acting in slab bending will be negligible for long shells, although the interaction with the end diaphragms or stiffeners should be provided for by an adequate amount and placing of steel in the manner described herein.

Longitudinal bending as a beam of curved cross section results in diagonal tension stresses in the shell. Furthermore, a transverse strip cut from the shell (Fig. 2) may be seen to act as a beam supported by the tangential shears on the cut sections. This transverse beam action develops transverse bending moments and thrusts. It is evident that inelastic buckling under the combined action of the longitudinal and transverse thrusts and moment must be prevented by proper proportioning of the structure.

Detailed discussion of the above action is presented below, followed by equations for use in the design of shells on the basis of ultimate load.

Longitudinal beam bending

Flexural stress—Referring to Fig. 2, it may be noted that at ultimate load the section at the point of maximum longitudinal bending moment is cracked below the neutral axis, with the concrete assumed to carry no tension. The longitudinal strain at the section of maximum moment is assumed to vary in direct proportion to the vertical distance from the neutral axis. For cylindrical shells, it is significant that a substantial portion of the concrete in compression is located in the region of high compressive strain, thereby permitting the use of the average compressive stress at ultimate load as being uniformly distributed over that portion of the shell above the neutral axis. Hognestad's idealized stress-strain curve[4] was used in determining the value of the average unit stress at yielding of the steel for establishing the relative position of the neutral axis. This procedure involves the assumption that $0.85 f_c'$ is the ultimate compressive stress of the concrete in the structure. Although this is consistent with past usage,[5] and may be considered as a desirable assumption from a conservative viewpoint, it has not been conclusively demonstrated that the strength of concrete in a structure of normal proportions is materially less than that of a cylinder cast and cured in the same manner. In fact, a recent series of unpublished tests,[6] for which all factors of fabrication were carefully controlled, has shown that virtually

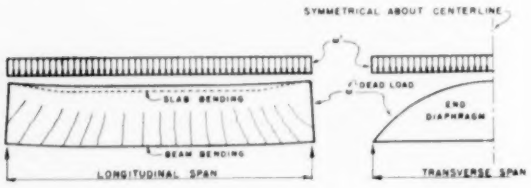

Fig. 1—Simply supported cylindrical shell

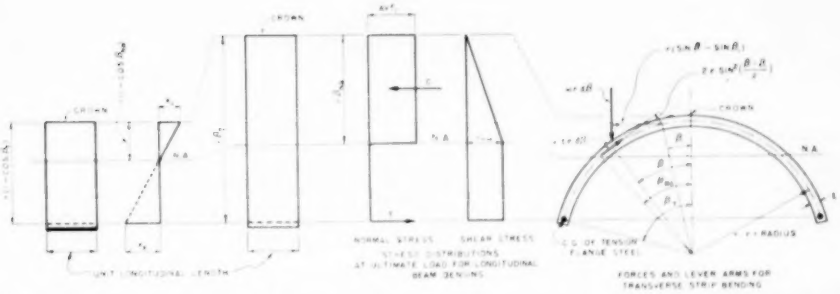

Fig. 2—Shell analysis for longitudinal beam bending and transverse strip bending for long thin shells

no difference exists between the ultimate strength of cylinders having a height-diameter ratio of 2 and prisms having a height-least dimension ratio of 5.3. This appears to be consistent with Hognestad's belief that the reduced value of strength can be attributed in part to differences in fabricating and curing cylinders as compared to the concrete in the structure.[4] Although the authors do not feel that the question of ultimate concrete strength is properly within the scope of this paper, they are of the opinion that $0.85 f_c'$ is the highest value that should be used until further information is available.

Diagonal tension—The distribution of unit shearing stress, which is used in the usual manner for diagonal tension stress, is assumed to be that resulting from the uniform distribution of the average compressive stress at failure and no tension in the concrete below the neutral axis as shown in Fig. 2. Due to the thinness of the shell as compared to a beam of normal proportions, no allowance is made for the shear that may be taken by the concrete. Shear tests of beams with webs varying from normal width to those of extreme thinness have shown a reduction in concrete effectiveness for decreasing web thickness.[7]

Transverse strip bending—Referring again to Fig. 2, a strip cut transversely from the shell will be acted upon by that portion of the total load carried by the strip and the tangential shear stresses developed by longitudinal beam bending in addition to the normal stresses resulting from flexure of the shell as a whole.

The transverse strip, therefore, acts as a beam loaded with that portion of the external vertical load applied to the strip and is supported by the difference between the tangential shearing stresses on the cut faces. Internal moments and stresses may be determined in the manner customary for beams. Lever arms and forces for the computation of the transverse strip bending moments in cylindrical shells are shown in Fig. 2.

Transverse thrust—A transverse strip is subjected to thrust acting on any radially cut section and varying from zero at either edge to a maximum value at the crown. The maximum transverse thrust, acting at the crown, is equal to the sum of the horizontal components of the tangential shear difference on either side of the crown.

Longitudinal slab bending

Long versus short shells—Bending of the shell as a slab in the longitudinal direction results in the development of radial and tangential shearing stresses with associated slab flexural stresses. If the longitudinal span is sufficiently long relative to the length of the shell arc, the load sustaining ability of the shell as a slab will be inappreciable and the resulting radial and tangential stresses may be considered negligible. In the case of shorter longitudinal spans, however, the radial and tangential stresses resulting from longitudinal slab bending may not be negligible, and a division of the applied load should then be made between longitudinal and transverse strips. A. L. L. Baker considers this problem for short shells for ultimate load theory, but does not define the dividing line between long and short shells.[8] Although, the transition from long to short shells is a gradual one,[3] a conservative limit may be set for long shells for the purpose of this paper by limiting long shells to those having values of r/L* less than 0.60 as in the case of elastic analysis.[9]

Continuity with end stiffener or diaphragm—Although the effect of longitudinal slab bending may be neglected for long shells insofar as contribution to the load carrying capacity of the shell is concerned, continuity between shell and end stiffener or diaphragm must be provided for by an adequate quantity and arrangement of steel if transverse cracking is to be avoided.

Insofar as elastic action is concerned, it has been shown by others that the slab moments resulting from continuity between shell and stiffener are essentially a local effect which is damped out within a relatively short distance from the stiffener.[3,9] That this is true also for loads beyond the elastic range is evident from the discussion of the exploratory tests that follow. For ultimate load design, slab moments at the junction with the stiffener can then be computed approximately from the radial components of the loads on longitudinal strips acting as free cantilevers having spans equal to the distance from the stiffener to the point of contraflexure of the slab. Assuming this distance to be a function of $\sqrt{rt}$, the exploratory tests described later suggest a multiplying factor of one for uniformly distributed loads.

INELASTIC BUCKLING

It has been shown by others that elastic buckling under maximum compressive stresses is not likely to be a critical factor at design loads for currently used proportions.[9] However, a considerably different situation exists in the case of ultimate load design.

If the design of the shell is to be such that failure occurs due to longitudinal beam bending of the shell as a whole, the compressive stress developed by such bending should not exceed the value determined by inelastic buckling theory for the shell proportions selected for use.[4]

Furthermore, at the crown the shell is subjected to a maximum transverse thrust combined with a maximum transverse bending moment. Therefore,

*See notation, p. 261, for definition of terms.

the proportions of the shell must also be selected to prevent inelastic buckling under combined bending and direct stress. The most effective tool for this purpose is the interaction diagram developed for such loading,[4,5] and is presented in nondimensional form for this case.

ULTIMATE LOAD EQUATIONS

Notation

β = Radian measure of angular position of points as indicated by subscript in Fig. 2

f_c'' = Ultimate compressive strength of concrete in shell, taken as 0.85 of the cylinder strength f_c', psi

f_{yp} = Yield point strength of steel, psi

v_{max} = Maximum unit shearing stress developed by longitudinal beam bending of shell, psi

v' = Difference in unit shearing stress for a transverse strip, psi

r = Radius of shell, in.

t = Thickness of shell, in.

L = Longitudinal span of shell, in.

w = Uniformly distributed dead load, psf of surface area

w' = Uniformly distributed live load, psf of horizontal projection

NA = Neutral axis

C, T = Total compressive and tensile force, respectively, developed by longitudinal beam bending, lb

ϵ_u = Ultimate unit compressive deformation for concrete taken as 0.0038

ϵ_s = Unit deformation of steel in tension flange

p = Steel ratio for tension flange, based on gross section of shell, *i.e.*, $A_s' = 2\,ptr\,\beta_T$, in which A_s' = sum of steel areas of both tension flanges

s = Perpendicular spacing of transverse or 45 deg inclined steel used for diagonal tension reinforcement, in.

A_s = Transverse steel area at crown

A_v = Area of steel used for diagonal tension reinforcement in spacing s, sq in.

V = Total vertical shear from longitudinal beam bending, lb

M_{max} = Maximum longitudinal beam bending moment, in.-lb

M_y = Continuity yield moment in shell at stiffener or diaphragm, in.-lb per ft of arc

M = Bending moment with subscripts T, TD, TL, and TV to denote transverse moments as follows: total, dead load (w), liveload (w'), and tangential shear difference, respectively, in.-lb per ft of shell length

W' = Total vertical load on transverse strip, lb per ft of shell length

T_{TC} = Transverse thrust at crown, lb per ft of shell length

T_{cr} = Critical inelastic buckling thrust, from tangent-modulus buckling curve, lb per ft of shell length

$\left(\frac{r}{t}\right)_o$ = Base value of (r/t) for nondimensional curves

$$\left(\frac{r}{t}\right)_o = \left[1.65\left(\frac{r}{L}\right)\left(\frac{1{,}800{,}000 + 460\,f_a''}{f_a''}\right)\right]^{2/3}$$

in which f_a'' is average unit stress on gross section of short (nonbuckling) length (see reference 4)

Longitudinal beam bending

Flexural equations—

$$\frac{M_{max}}{tr^2} = 2\,f_{yp}\,p\,\frac{\beta_T}{\beta_{NA}}\,(\sin\beta_{NA} - \beta_{NA}\cos\beta_T) \qquad (1)$$

$$p = \frac{\text{Avg}\,f_c\,\beta_{NA}}{f_{yp}\,\beta_T} = \frac{2}{3}\,\frac{\max f_c\,\beta_{NA}}{f_{yp}\,\beta_T}\ \text{(approx.)} \qquad (2)$$

The above equations may be used up to the balanced reinforcement in the tension flanges. In view of the high diagonal tension stresses developed in

the shell, the development of balanced reinforcement or greater does not appear to be a realistic possibility. Recommended value for $\beta_{NA}/\beta_T = 0.3$.

Diagonal tension equations—

$$v_{max} = \frac{V}{2\,tr}\left(\frac{\beta_{NA}}{\sin\beta_{NA} - \beta_{NA}\cos\beta_T}\right) \quad (3)$$

$$S = \frac{f_{vp}\,A_v}{t\,v_{max}} \quad (4)$$

No allowance has been made for the shear that may be taken by the concrete due to the thinness of the shells compared to a beam of normal proportions.

Transverse strip bending equations—

$$M_T = M_{TD} + M_{TL} + M_{TV} \quad (5)$$

$$M_{TD} = -\frac{w\,r^2}{12}\left[(\beta_1 - \beta_T)\sin\beta_1 + \cos\beta_1 - \cos\beta_T\right] \quad (6)$$

$$M_{TL} = -\frac{w'\,r^2}{24}(\sin\beta_T - \sin\beta_1)^2 \quad (7)$$

$$M_{TV} = V'_{max}\,tr^2\left((\beta_T - \beta_1) - \sin(\beta_T - \beta_1)\right) \qquad \beta_1 > \beta_{NA} \quad (8)$$

$$V'_{max} = \frac{W'}{2\,tr}\left(\frac{\beta_{NA}}{\sin\beta_{NA} - \beta_{NA}\cos\beta_T}\right) \quad (9)$$

Transverse thrust equations—

$$T_{TC} = \frac{V'_{max}}{\beta_{NA}}\,tr\,(\beta_{NA}\sin\beta_T + \cos\beta_{NA} - 1) \quad (10)$$

V'_{max} = see Eq. (9)

Longitudinal slab bending

Continuity moment equation—

$$M_y = -\frac{rt\cos\beta_1}{24}(w + w'\cos\beta_1) \quad (11)$$

Inelastic buckling

Longitudinal beam bending compression—

$$\left(\frac{r}{t}\right)_{max} = \frac{250{,}000 + 54.3\,f_c'}{f_c'} \quad (12)$$

A tangent-modulus buckling curve was developed in accordance with pp. 43-44 of reference 5 to establish that virtually the full value of f_c'' could be assured at failure if the ratio of (r/t) to $(r/t)_o$ did not exceed 0.2. Eq. (12) is based upon the value of 0.2.

Combined transverse thrust and moment—Fig. 3a and 3b were developed from the theory presented in reference 5 for the particular shell analyzed as an example at the close of this paper. Similar curves can be constructed for other steel contents to provide flexibility for design as well as review. Fig. 3a, in nondimensional form, would cover all steel contents, but additional curves such as in Fig. 3b are required for other steel contents. The use of these curves are illustrated in the example.

EXPLORATORY TESTS

Three test shells were constructed of such size as to permit testing under controlled loading in a testing machine. The shells were of the dimensions given in Fig. 4, and were tested under vertical loads uniformly distributed in the transverse direction at the third-points of the longitudinal span. Fig. 4 and 5 illustrate the manner of loading as well as the method of measuring vertical and horizontal deflections at midspan of each tension flange. Electrical resistance strain gages were cemented to the external surface of the shells in such a manner so as to obtain the vertical distribution of longitudinal strains at the midspan transverse section and also the distribution of longitudinal strains at the crown in the vicinity of the diaphragms.

The steel content in the shell proper was identical for all shells and consisted of a double layer of 1 in. x 1 in. x 15-gage welded wire mesh in the central 36 in. of the shell, but only a single layer in the outer 12 in. on each end. The double layer served the purpose of providing more than adequate reinforcement against transverse bending failure in the regions of greatest shear difference at the loads. The mix was a portland cement and sand mortar having an average compressive strength of 5700 psi at time of testing of the shells.

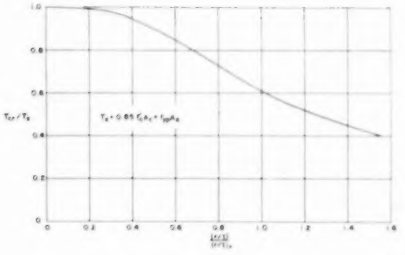

Fig. 3a—Approximate tangent-modulus buckling curve for critical transverse thrust

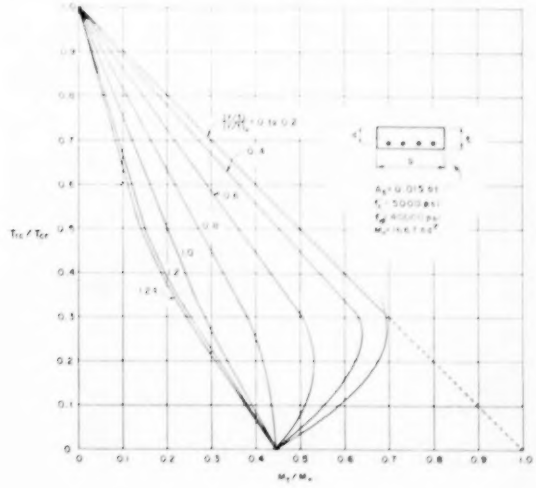

Fig. 3b—Interaction diagram for transverse thrust and moment

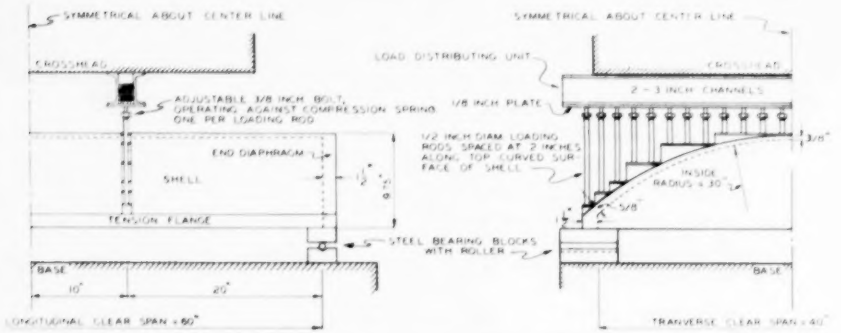

Fig. 4—Shell dimensions and loading method

Continuity between the shell and each end diaphragm was provided by bending the 15-gage mesh that reinforced the diaphragms and extending it several inches into the shell. Each diaphragm also contained ties between the tension flanges at each end consisting of one ½-in. and one ¼-in. round bars.

The steel was varied in the tension flanges of the three shells with the expectation that the most lightly reinforced would reach the yield point. However, all three shells failed due to diagonal tension in the outer thirds of the shell. The principal steel in each tension flange of shell No. 3 consisted of one ¼-in. round bar. The steel content in the flanges of shells No. 2 and 1 was twice and four times, respectively, that of shell No. 3.

To prevent failure from bond, the bars in the tension flanges, and also the tie bars in the diaphragms, were extended beyond the concrete and anchored by nuts on the threaded ends (Fig. 5).

The results from shell No. 3, in which the tension flange steel reached the highest strain, are discussed below, and are typical of the other two shells.

Longitudinal beam bending

Longitudinal strains—One of the assumptions of the ultimate load theory for shells is that the longitudinal strains resulting from longitudinal beam

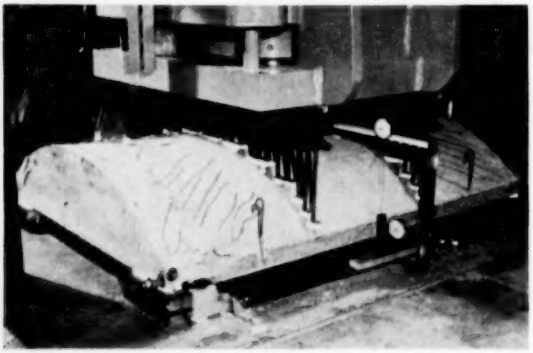

Fig. 5—Method of measuring midspan deflections

bending vary in direct proportion to the vertical distance from the neutral axis at ultimate load. The average strains obtained from longitudinal gages placed from flange to flange on the top surface at midspan of shell No. 3 are shown in Fig. 6. The first visible crack occurred in one tension flange at 10,000 lb, and tensile cracking in the other flange became evident at 12,000 lb at which load an approximately straight line variation of strains developed with failure at 12,500 lb. The maximum measured strain at the level of the steel in the tension flanges of shell No. 3 was 470 micro-in. per in., or approximately 14,100 psi in the steel, neglecting the effect of the concrete.

Increased flange steel area, without change in quantity of other reinforcement, only slightly increased the capacity of the shells. The ultimate load for shell No. 1 was 15,860 lb and that for shell No. 2 was 14,800 lb, for steel areas of four and two times, respectively, that of shell No. 3 which failed at 12,500 lb. As in the case of beam tests, the increase may be attributed to the reduction in tensile strain due to the greater steel area, and hence a lesser deflection for the same load, thereby slightly increasing the ultimate capacity.

It should be noted that the increase in strain in the tension flange was virtually the same for equal load increments up to 6000 lb. It is not unreasonable to conclude that, beyond the tensile strain of 280 micro-in. per in. at the 6000-lb load, readjustment of strain due to tensile cracking affected the gage recordings even though the first visible cracks occurred at 10,000 lb. The strain of 0.00028 at the 6000-lb load is a reasonable maximum tensile strain for concrete.

Diagonal tension—All three shells failed in diagonal tension, the crack pattern of each being practically identical to that illustrated in Fig. 7. The paths of diagonal tension failure, as identified by the cracks, are unusually well defined in direction and position. In general, the cracks are quite typical of those developed in the deep beams having thin webs that were tested for high shearing stresses during World War I.[7]

As in the case of the beams of reference 7, the shells developed much higher shearing stresses than in beams having only nominal amounts of web reinforcement in combination with lower depth to span ratios and less positive

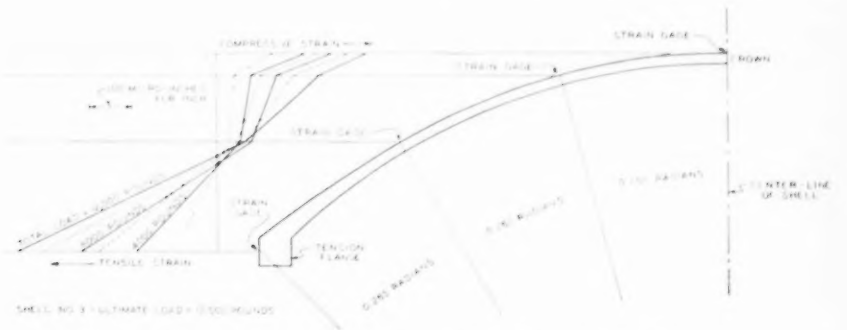

Fig. 6—Distribution of strains resulting from longitudinal beam bending

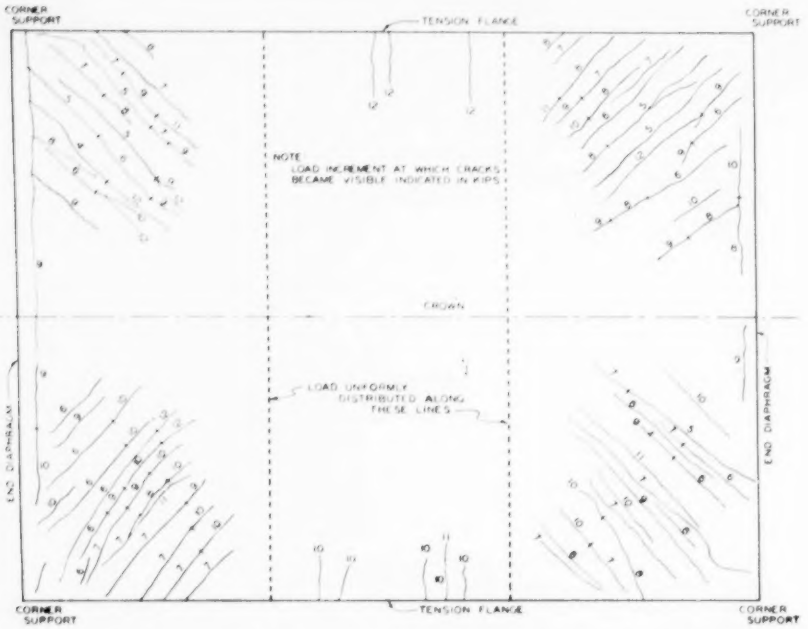

Fig. 7—Developed plan view of shell No. 3, showing crack pattern. This diagonal tension failure occurred under an ultimate load of 12,500 lb

anchorage against bond failure. Table 1 contains values of v_{max} at the load increment just prior to failure as computed from Eq. (3) and (4), and are quite comparable to the shearing stresses given in reference 7.

It is clearly evident, from the agreement between the results of the two equations, that the mesh acting as diagonal tension reinforcement had reached yielding. In computing the values from Eq. (4), the minimum (55,000 psi) and maximum (64,000 psi) values of f_{yp} were obtained from tension tests of the wire mesh. The position of the neutral axis for use in Eq. (3) was calculated from the strains measured on the crown and tension flanges at the last load increment prior to failure in each case inasmuch as the shell had not failed by longitudinal beam bending moment and Eq. (1) would not apply.

TABLE 1—COMPUTED LOAD INCREMENT JUST PRIOR TO FAILURE

Shell No.	V, lb	v_{max}, psi		
		Eq. (3)	Eq. (4)	
			Minimum f_{yp}	Maximum f_{yp}
1	7930	1460	1190	1390
2	7400	1310	1190	1390
3	6250	1260	1190	1390

Diagonal tension appears to be one of the more critical of the various stresses developed in shells, and more particularly because of the limited room for protective steel and the increased need for positive anchorage of all tensile reinforcement against bond failures.

Transverse strip bending—The action of transverse strip bending is evidenced by the inward lateral deflections that occurred in the range of loading prior to the first diagonal tension crack (Fig. 9). This is consistent with elastic response to symmetrical loadings uniformly distributed in a transverse direction.[9] Early failure from diagonal tension prevented a more definite identification of such action in the inelastic range.

Longitudinal slab bending

Continuity strains—Gages for the measurement of continuity strains in the vicinity of the diaphragms were placed only on shells No. 2 and 3. The results are shown in Fig. 8 for the last load increment prior to failure. It may be noted that even though yield point strain for the steel must have been substantially exceeded just inside the diaphragm, the steel was just about at the yield point within a very short distance from the diaphragm, indicating that a substantial amount of negative moment was still being carried by slab bending. The uncertainty of the actual position of the 15-gage wire in a $\frac{3}{8}$-in. shell thickness prevents any rational attempt to compute the magnitude of the moment.

However, it is evident that the moments are damped out within a relatively short distance from the stiffener or diaphragm even for the rather severe third-point loading, as shown in Fig. 8. In the case of loads distributed uniformly in the longitudinal direction as well as the transverse direction, the point of contraflexure for the slab will shift toward the stiffener. For these

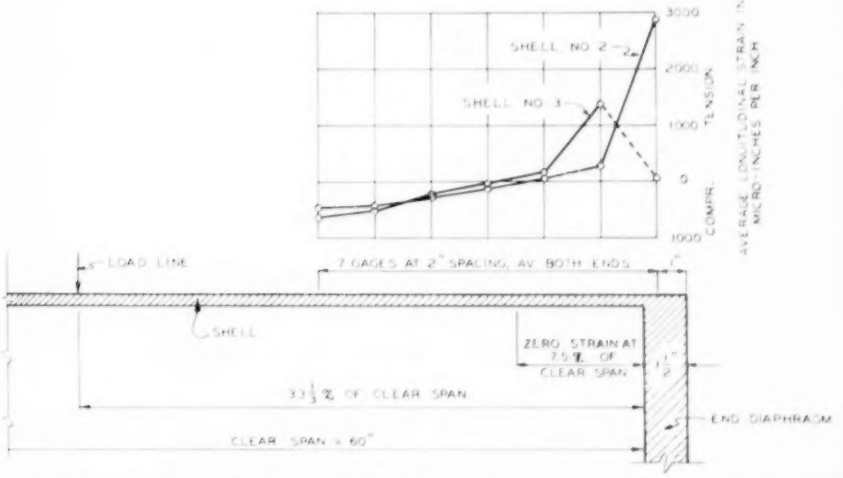

Fig. 8—Top surface strains in vicinity of end diaphragm at 94 percent or more of ultimate load

tests, a transverse section at a distance $\sqrt{rt}$ from the stiffener could be safely assumed to have no moments or radial shears resulting from such continuity. From this transverse section, slab moments at the stiffener may be calculated and steel determined therefrom.

Deflections

The deflection curves of Fig. 9 show a behavior of the shell consistent with

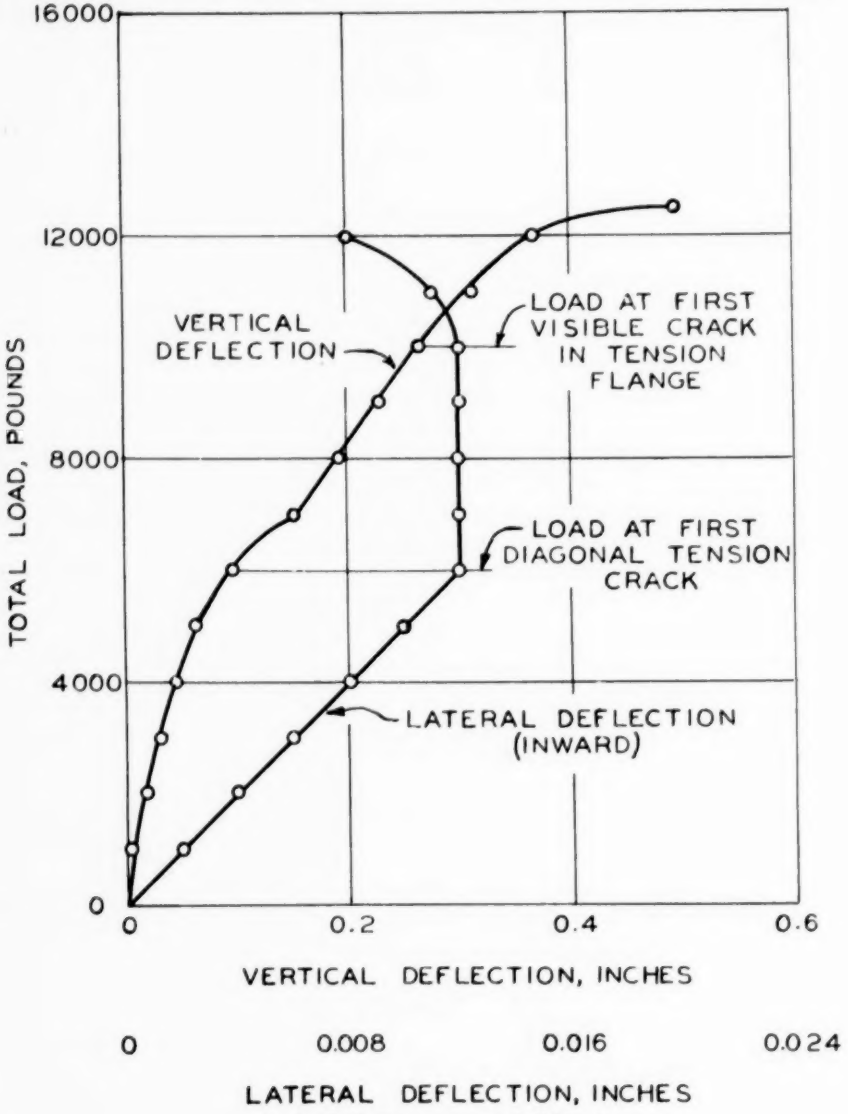

Fig. 9—Deflection curves for shell No. 3

elastic theory up to the development of the first visible crack. The vertical deflection was affected more by the cracks in the tension flanges than by those resulting from diagonal tension, even though failure was due to the latter. The change in lateral deflection was immediate and large for both types of crack formation.

SUMMARY

The structural behavior of thin-shell roofs at ultimate load may be divided into the longitudinal bending of the shell as a beam of curved cross section and the longitudinal bending of the shell as a slab. Insofar as long shells are concerned, the factors to be observed in design are:

1. *Longitudinal beam bending*, with resulting flexural stresses, diagonal tension, transverse moment, and transverse thrust

2. *Longitudinal slab bending*, with regard to continuity at end stiffener or diaphragm

3. *Inelastic buckling*, with regard to proportioning for full development of compressive strength under longitudinal thrust and adequate reinforcement against combined transverse thrust and moment

EXAMPLE

The following example has the same dimensions as Example 1 in reference 8. However, in determining steel content by ultimate load equations, a live load of three times that used in reference 8 has been selected.

Dimensions	Loads and stresses
r = 372 in. or 31 ft	Dead load = 47 psf
t = 3.75 in.	Live load = 75 psf
L = 744 in. or 62 ft	f_c' = 5000 psi
β_T = 37 deg. = 0.64577 radians	f_{yp} = 40,000 psi

It has been assumed that the center of gravity of the tensile steel lies 3 deg above the longitudinal edge of the shell in each tension flange, the total angle from crown to edge being 40 deg.

Tension flange steel—

$$M_{max} = \frac{(43.3 \times 47 + 39.8 \times 75)\ (62)^2}{8} \times 12 = 29{,}000{,}000 \text{ in.-lb}$$

$$56 = 2(40{,}000)\ p \frac{0.6458}{\beta_{NA}} (\sin \beta_{NA} - 0.7986\ \beta_{NA}) \quad \text{(Eq. 1)}$$

from which, for $\beta_{NA}/\beta_T = 0.3$
$p = 0.00556$, $A'_s = 10.02$ sq in., $\beta_{NA} = 11$ deg 6 min $= 0.19373$ radians
Use 9–#7 bars at 3-in. centers from longitudinal edge; continue #4 bars at 9-in. centers to crown.

$$\text{Maximum } f_c = \frac{3}{2}\left(\frac{40{,}000}{0.3}\right) 0.00556 = 1110 \text{ psi} \quad \text{(Eq. 2)}$$

Diagonal tension steel—

$$V_{max} \text{ at support} = \frac{5025\ (31)}{2\ (3.75)\ (372)}\left(\frac{0.1937}{0.1925 - 0.1937 \times 0.7986}\right) = 287 \text{ psi} \quad \text{(Eq. 3)}$$

$$S = \frac{40{,}000\ A_v}{3.75\ (287)} = 37\ A_v \quad \text{(Eq. 4)}$$

Use #4 bars at 7½ in. from support to ¼-point, increase to 15 in. from $L/4$ to $L/2$.

Transverse bending moment at crown—

$$M_{TD} = -\frac{47\ (372)^2}{12}\ (0 + 1 - 0.7986) = -109{,}000 \text{ in.-lb per ft} \quad \text{(Eq. 6)}$$

$$M_{TL} = -\frac{75\ (372)^2}{24}\ (0.6018)^2 = -156{,}700 \text{ in.-lb per ft.} \quad \text{(Eq. 7)}$$

$$V'_{max} = \frac{5025}{2\ (3.75)\ (372)}\left(\frac{0.1937}{0.1925 - 0.1937\ (0.7986)}\right) = 9.25 \text{ psi} \quad \text{(Eq. 9)}$$

$$M_{TV} = 9.25\ (3.75)\ (372)^2\ (0.0441) = 212{,}000 \text{ in.-lb per ft} \quad \text{(Eq. 8)}$$

$$M_T = -109{,}000 - 156{,}700 + 212{,}000 = -53{,}700 \text{ in.-lb per ft} \quad \text{(Eq. 5)}$$

Transverse thrust at crown—

$$T_{TC} = \frac{(9.25)\ (3.75)\ (372)}{0.1937}\ (0.1937 \times 0.6018 + 0.9813 - 1) = 6520 \text{ lb per ft.} \quad \text{(Eq. 10)}$$

Transverse steel at crown—

$$\frac{r}{t} = 99, \quad \left(\frac{r}{t}\right)_a = 80, \quad \frac{(r/t)}{(r/t)_a} = 1.24$$

from Fig. 3a: (assuming $A_s = 0.015bt$) $T_a = 191{,}200 + 27{,}000 = 218{,}000$ lb per ft

$$\frac{T_{cr}}{T_a} = 0.51 \text{ for } \frac{(r/t)}{(r/t)_a} = 1.24, \therefore T_{cr} = 111{,}000 \text{ lb per ft}$$

for Fig. 3b: $\dfrac{T_{TC}}{T_{cr}} = \dfrac{6520}{111{,}000} = 0.059$

from Fig. 3b: $\dfrac{M_T}{M_a} = 0.41$ for $\dfrac{(r/t)}{(r/t)_a} = 1.24$ and $\dfrac{T_{TC}}{T_{cr}} = 0.059$

$M_a = 151{,}200$ in.-lb per ft, from which maximum $M_T = 0.41\ (151{,}200) = 62{,}000$ in.-lb per ft

62,000 in.-lb per ft > actual M_T of 53,700 in.-lb per ft, and therefore, $A_s = 0.015bt = 0.675$ sq in. per ft is somewhat greater than is required. Use #5 bars at 6 in. centers.

Continuity steel at crown—

$$M_y = \frac{372\ (3.75)}{24}\ (47 + 75) = 7100 \text{ in.-lb per ft} \quad \text{(Eq. 11)}$$

$A_s = 0.0693$ sq in per ft from reference 4 or C. S. Whitney's plastic theory

Use #3 bars at 12-in. centers throughout arc

Inelastic buckling

$$\left(\frac{r}{t}\right)_{max} = \frac{250{,}000 + 271{,}500}{5000} = 104 \quad \text{(Eq. 12)}$$

$\left(\dfrac{r}{t}\right)_{actual} = 99 < 104$, which is satisfactory

REFERENCES

1. Baker, A. L. L., "A Plastic Design Theory for Reinforced and Prestressed Concrete Shell Roofs," *Magazine of Concrete Research* (London), No. 4, July 1950.

2. Johansen, K. W., "Critical Notes on the Calculation and Design of Cylindrical Shells," *Final Report of the Third Congress*, International Assn. for Bridge and Structural Engineering, 1948.

3. Lundgren, H., "Cylindrical Shells—V. 1," Danish Technical Press, The Institution of Danish Civil Engineers, Copenhagen, Denmark, 1949.

4. Hognestad, E., "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," *Bulletin* No. 399, Illinois Engineering Experiment Station, 1951.

5. "Inelastic Buckling of Plain and Reinforced Concrete Columns, Plates, and Shells," *Bulletin* No. 3, Nebraska Engineering Experiment Station, 1953.

6. Unpublished tests by Professor J. S. Blackman, Department of Engineering Mechanics, University of Nebraska.

7. Slater, W. A., Lord, A. R., and Zipprodt, R. R., "Shear Tests of Reinforced Concrete Beams," *Technologic Paper* No. 314, National Bureau of Standards, 1926.

8. Baker, A. L. L., "Ultimate Strength Theory for Short Reinforced Concrete Cylindrical Shell Roofs," *Magazine of Concrete Research* (London), No. 10, July 1952.

9. "Design of Cylindrical Concrete Shell Roofs," *Manual of Engineering Practice* No. 31, American Society of Civil Engineers, pp. 58, 97-99.

Title No. 51-13

"Accidental" Air in Concrete*

By M. F. MACNAUGHTON† and JOHN B. HERBICH‡

SYNOPSIS

This paper deals with instances where "accidental" air in concrete§ was found to be present in amounts far beyond those normally encountered in plain mixtures. The authors list a number of such cases which have come to their attention in recent years in the area in Canada covered by the Laurentian shield. In connection with one specific instance an outline is presented of the investigation of the problem and the measures adopted to control vagrant and final air content in the concrete mixtures on the work within reasonable and normally acceptable limits.

INTRODUCTION

This paper presents a special problem in regard to "accidental" air entrainment§ in concrete mixtures which, apparently, has escaped the attention of nearly everyone connected with the manufacture of concrete in eastern Canada, and, possibly, in other parts of the continent.

The first indication of the existence of such a situation arose when a report was made on a sample of gravel aggregate from St. Johnsbury, Vt., for the Canadian Pacific Railway Co. in 1937. This gravel was being considered for use in the construction of a railroad scale pit. The gravel was suitably graded for use as concrete aggregate and the organic matter content was apparently low. A trial mixture prepared with this gravel showed low density, with a unit weight of 137 lb per cu ft, and a compressive strength of only about 50 percent of what was expected from the *W/C* used. A direct quote from the report on this subject is made here:

> For some reason which we could not determine the sand has, when surface dried, the peculiar property of holding a film of adsorbed air so tenaciously that it is not readily displaced by water and the surface of the material wetted. As a result of this characteristic, if the sand is at all dry, a great deal of air is incorporated into the concrete mixture in which it is used and no amount of mixing seems to be effective in displacing these air films. The concrete is light in weight, is filled with air bubbles, and, for the cement content used, has a very low compressive strength, being only

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§"Accidental air" is a new term in concrete technology. It may be defined as air accidentally entrapped or incorporated into a concrete mix either mechanically or by any other means. It may be unstable in nature and vary in size from microscopic to $\frac{1}{4}$ in. or larger air bubbles. The term "accidental air" should not be confused with the term "entrained air" which is generally used to describe the air introduced to the concrete mix in the form of microscopic air bubbles by addition of various air-entraining agents.

905 psi at 7 days, as compared with a normal strength of about 2000 psi for similar concrete made with normal aggregates.

As a result of the tests on this sample, the Canadian Pacific for its work at St. Johnsbury obtained aggregates from other sources which behaved normally in concrete mixtures.

The conclusions in respect to this gravel were, as it was later discovered, partially wrong, but basically they were correct. Air was being entrained, though nothing was known about air entrainment in those days, and the excessive amount of air entrainment, as indicated by the low weight of the resultant concrete, is today a perfectly reasonable explanation for the low strength obtained from this trial mixture.

The next recorded contact with such a situation came in 1947, when some tests were carried out for the Great Lakes Power Co., Ltd., in connection with a project by which they proposed to raise the level of a dam north of Sault Ste. Marie in Ontario. In this case the designing engineers in the United States had specified the use of an air-entraining agent to entrain 3–5 percent of air in the concrete mixture. Trial mixtures with the aggregates submitted for test showed that plain mixtures, without the addition of an air-entraining agent, entrained or entrapped about 6 percent of air. As in the case of the Canadian Pacific Railway, this source of supply was abandoned, and aggregate was obtained from another source which showed no evidence of abnormal amounts of entrained air in plain mixtures.

Nothing more developed until, in 1949, the Shawinigan Engineering Co., Ltd., employed Milton Hersey Co., Ltd., as concrete consultants on the work on the power dam on the St. Maurice River at La Trenche. Here it was found that the only fine aggregate available within reasonable distance of the work had the power of attracting accidental air into the concrete in the amount of 2.0 to 3.5 percent. In this particular case, since the coarse aggregate was high in specific gravity (2.90 average) and the loss in weight, from a point of view of stability, was of no consequence, it was recommended that a small amount of an air-entraining agent be added to the mix to introduce some stable air which would be resistant to freezing effects, and the specifications were changed to permit an air content in the exposed concrete of 5–7 percent in the aggregate fraction below 1½ in.

During the next two years a number of cases were encountered where natural aggregates yielded mixtures with excessive amounts of air, but it was possible to dispose of them by utilizing aggregates from other sources, or, in the case of structural concrete, where comparatively slight variation in weight is not an important factor in design, by adding a small amount of an air-entraining agent to provide some stable air content.

PERIBONKA PROJECTS

Early in 1951 the Aluminum Company of Canada, Ltd., called on the services of Milton Hersey Co., Ltd., as inspectors on the concrete work on their two projects on the Peribonka River, involving a total of approximately

800,000 cu yd of concrete. The first job was to locate a suitable source of supply of fine aggregate. Sand in that area is abundant. The first tests of sand, from a deposit located about a mile from Chute du Diable, indicated that the natural sand possessed abnormal capacity in entraining air in plain mixtures, in some cases as high as 9 percent. It should perhaps be mentioned here that the air content was measured on the $-1\frac{1}{2}$-in. fraction of the concrete. In the case of a concrete sample with maximum size of aggregate of 6 in., the stone above $1\frac{1}{2}$ in. was removed by hand and the remaining sample tested in an air meter by the approved method.* On learning about these unusual results the Aluminum Company of Canada decided to continue the survey until a suitable supply of sand was found.

Before the investigation was completed, all deposits accessible by road within a radius of 30 miles were sampled and tested, as well as a number of deposits beyond that radius. Some 2600 test holes were put down and the final conclusion was that all of the sand in the area had the same characteristic, in greater or lesser degree, of attracting air into the mixture. The characteristics of the sands were as follows:

Coarse sand: Average value from Sept. 1, 1951, to May 31, 1952
Fineness modulus: 2.61
Organic matter content (by sodium hydroxide color test): 100 parts per million

Grading: Retained on	
No. 8 mesh	4.0 percent
No. 16 mesh	19.1 percent
No. 30 mesh	53.5 percent
No. 50 mesh	88.1 percent
No. 100 mesh	96.5 percent

To supplement the deficiency in fine fractions, a blending sand was added to the coarse sand.

Fine sand: Average value from Sept. 1, 1951, to May 31, 1952
Fineness modulus: 0.88
Organic matter content: Nil

Grading: Retained on	
No. 8 mesh	0.2 percent
No. 16 mesh	0.7 percent
No. 30 mesh	2.4 percent
No. 50 mesh	14.3 percent
No. 100 mesh	69.9 percent

The grading of the average blended sand was 90 percent coarse and 10 percent fine sand.

Organic matter

An investigation at McGill University by Dr. H. W. Lucien showed that the air-entraining properties of this sand were due to the presence of a considerable proportion of organic matter in the sand. This organic matter seems to be derived from the decomposition of organic matter in clay beds underlying the sand deposits. Some of the volatile products of the organic decomposition have been recondensed on the surfaces of the sand grains overlying the clay deposits and have imparted special properties to the sand as far as its use in concrete mixtures is concerned.

*Generally a Press-Ur-Meter air meter was used for air content determinations. Occasionally an Ayr-Meter was used for checking purposes.

Dr. Lucien was able to separate the organic matter present in the sand into three main groups. The first of these, which was a nonvolatile brown crystalline solid, possessed foaming properties when used in concrete mixtures. The second, which consisted of a liquid with a high vapor pressure, exhibited foaming properties to an even more marked degree. The third constituent consisted of methane gas which apparently was adsorbed on the surface of the sand particles, or, possibly, dissolved in the second constituent mentioned above.

All could be removed by cold solution in ethyl ether. By distillation of the solution at 0 C, and reduced pressure, the ether was removed along with the methane. The extract remaining consisted of a mixture of components 1 and 2, and by heating on a steam bath component 2 was volatilized, leaving a residue of the brown crystalline solid.

The extracted sand possessed no air-entraining or foaming properties, but re-introduction of the extracted components resulted in renewal of the foaming effects.

Long continued aeration of the sand apparently results in virtual elimination of components 2 and 3, with appreciable reduction in foaming or air-entraining properties, but does not eliminate component 1, which is comparatively nonvolatile. Heating to 212 F, such as occurs in heating fine aggregate with live steam in winter operations, assists in eliminating the volatile constituents but is only partially effective due to the difficulty of eliminating the volatile constituents from a comparatively large mass of sand in a stockpile. Heating to still higher temperature in a drier would effectively remove the greater part of the surface-activating materials, but such a procedure, apart from the cost, would present serious problems in mass concreting operations because of the necessity of after-cooling.

These organic products, derived from the anoxemic decomposition of organic matter, behave quite differently from normal organic materials which contaminate sand as a result of the decomposition of organic matter at the surface in the presence of air. The ordinary sodium hydroxide color test is useless, as a concentration of as high as 500 parts per million may show a color corresponding to No. 1 on the color chart. At or near the surface, where aeration and oxidation have occurred, the sand has little or no air-entraining property; the effect increases with depth, as might be expected. The effect varies widely with sand from different parts of the same deposit. In the first trial mixtures accidental air contents of 5–9 percent in plain mixtures were obtained. Later even higher air contents were observed and, in one instance, when an investigation was underway to determine why there was unusual difficulty in controlling the air content of the concrete, a trial mixture of plain concrete was prepared in the laboratory with sand which was then being used in the plant, which contained 23 percent of accidental air. This concrete weighed 118 lb per cu ft and had a consistency like that of whipped cream.

Laboratory tests

Once the problem was understood the remedy was fairly simple. Since the air inclusion resulted from the presence of a foaming agent in the sand the obvious remedy was to eliminate this excess air by the addition of a foam control agent. At the same time, since a certain amount of properly entrained air is desirable in concrete to afford protection against freezing effects, the foam control agent should be not too drastic in its effect and should leave a residue of stable air in the mix.

By investigation on a laboratory basis, prior to the start of the actual construction, which involved testing the effect of about 40 different foam control agents, several agents were found which functioned in a satisfactory manner. Most of the satisfactory foam control agents belonged to the higher alcohol group and some of these could be used commercially in concrete mixtures at no great cost.

Preliminary field experience

The first approach to the problem was to use the foam control agent to reduce the air in the mixture, as discharged from the mixer, to the desired amount. It was found that this approach did not give adequate control since variations in air-entraining characteristics of the sand were quite marked; and it was difficult, in plant control, to keep pace with such changes. From the start of concrete operations on these projects, mortar samples, including all aggregate below the ½-in. screen size, were prepared and subjected to freezing resistance tests. Some of the early tests indicated that even 5 percent of air retained in the − 1½-in. fraction under such conditions did not give complete protection against the effects of 150 cycles of the freezing test. It appeared that not all of this residual air was effective in providing protection against freezing effects. This was supported by results of laboratory tests which indicated that, in some cases, from 7 to 9 percent of accidental air was necessary to obtain adequate protection against freezing effects.

A further difficulty was that the accidental air introduced by the organic matter in the sand was comparatively unstable, and the resultant foam broke down quite rapidly. As a result, concrete might be discharged from the mixer in a perfectly workable condition to a transport vehicle, and by the time it arrived at the point of placement so much air had been lost that at the delivery point the concrete was harsh and unworkable, and, incidentally, contained insufficient air to protect it against freezing effects.

Pumped concrete operations

This loss of air was particularly objectionable in pumped concrete operations, where satisfactory performance depends entirely on the use of a mixture which will not segregate in the line. With an unstable air content, sufficient loss of air to the top of the line will result in an open air channel along the top of the line and a mechanical "freeze" of the main bulk of the concrete in the line. In such a case it becomes necessary to stop operations and empty the line. With a line 1000 ft long this means possibly a 4-hr delay and the loss

of 10 cu yd of concrete in the line plus what is in transit to the pumping machine from the mixer plant.

Before the concrete work had proceeded far along these lines it was realized that, regardless of all else, concrete discharged from the mixer had to have a residual content of stable entrained air. This was achieved by adding an excess of air-entraining agent to the mixture to introduce an excess of stable air into the mixture, and then adding a sufficient amount of the foam control agent to eliminate excess air and reduce the air content, on discharge from the mixer, to the desired amount. By doing this it was felt that the residual air in the mixture would be fairly stable and would remain reasonably constant in amount from discharge from the mixer to the point of final placement in the forms.

This was found to be so, and it was possible to discharge concrete from the mixer with an average air content of 5–7 percent, haul it 3 miles in Dumperete trucks, put it through a Pumperete machine, pump it 800 ft, and discharge it from the end of the line with an average air content of 3.5–5.5 percent in the $-1\frac{1}{2}$-in. fraction of the mix.

Field control

No great extra cost was involved in this operation. Air-entraining agents are comparatively cheap, and, with a sand such as described, the amount required to introduce an extraordinary amount of air is comparatively small. Foam control agents are more expensive, but the amount required is extremely small. For the Peribonka work the amount used varied from 1 to 15 cc per cu yd of concrete, the maximum being used when, for a short period, the sand supplied to the mixing plant yielded 23 percent of air in a plain mixture.

Adequate air control under such conditions calls for the continual presence of an inspector on the hopper floor of the mixing plant who does nothing but run air tests on the concrete discharged from the mixers. From the results of these tests the amount of foam control admixture is varied to insure a constant content of stable air in the concrete.

After establishing this practice it was possible to obtain much more satisfactory results from the test samples submitted periodically for freezing and thawing tests. Of the last batch of 50 samples, covering an operational period of approximately 2 months, not one showed any damage as a result of 150 cycles of the freezing test.

In the first approach to this problem, in June, 1951, little was known about it; the difference between the behavior of accidental and entrained air in concrete was not fully recognized. At that time it was felt that if between 3 and 6 percent of air was obtained in the final concrete, the specifications were satisfied as far as frost resistance was concerned.

On June 5, 1951, the observations shown in Table 1 were made by the inspectors on discharge of the concrete from the mixers and on final delivery at the forms from the Pumperete machine. At this time the 8-in. diameter Pumperete line was about 800 ft long.

TABLE 1—CONCRETE OBSERVATIONS (JUNE 15, 1951)*

At the mixer			At the forms		
Time	Slump, in.	Air content, percent	Time	Slump, in.	Air content, percent
P.M.			P.M.		
3:00	1	6.0	3:00	1½	4.5
3:10	1	7.6			
			3:15	½	4.5
			3:30	1	4.9
3:45	2½	7.5	3:45	1¼	4.0
			4:00	1	3.8
4:05	2½	6.0			
			4:15	1½	4.9
4:30	2½	7.0			
			4:35	1¼	3.9
			4:45	1	4.7
5:00	1¼	6.4	5:00	1	4.9
			5:15	1	4.0
5:17	1½	5.2			
			5:30	¾	4.7
5:45	5	7.5	5:45	1	4.8
6:00	2½	5.8			
			6:10	4	4.5
6:25	—	6.4	6:45	1½	3.7
Maximum	5	7.6	Maximum	4	4.9
Minimum	1	5.2	Minimum	½	3.7
Average	2¼	6.5	Average	1¼	4.4

*Admixtures: air-entraining agent + foam control agent.
About 20 min elapsed between the time when concrete left the mixing plant and was placed in the forms.

Final air control seemed to be fairly satisfactory and within normal limits, but freezing resistance test results on samples taken from similar concrete within the next week or two showed some variations from the properties desired.

Table 2 shows the behavior of 21 specimens of concrete taken during this period. While 16 of the 21 show no visible damage after 150 cycles of the freezing resistance test, four show slight scaling and one shows severe damage. All of the damaged specimens were from concrete mixtures containing from 4 to 5 percent air in the $-1\frac{1}{2}$-in. portion of the mixture and should have been resistant to the freezing test.

Up to this time the amount of residual air in the concrete was controlled by simply adding enough foam control agent to lower the final air content, on discharge from the mixer, to about 2 percent above the air content that

TABLE 2—RESISTANCE TO 150 CYCLES OF FREEZE-THAW TEST*

Degree of scaling†	0	1	2	3	4	5
Percent of air						
2	1					
3	3					
4	5	3		1		
5	5	1				
6	1					
7	1					
Total number of tests	16	4	0	1	0	0
Percentage of the total number of tests	76.2	19.0	0	4.8	0	0

*The testing procedure was as follows. Two-in. cubes were prepared from concrete samples with aggregate over ½ in. in size removed by wet screening. The cubes were moist cured for 28 days, then allowed to air dry in the laboratory for 7 days. At 35 days they were immersed in water and subjected to alternate cycles of freezing at −20 F and thawing at +70 F for a total of 150 complete cycles.
†Degree of scaling ranges from 0 to 5, where 0 indicates no scaling and 5 complete disintegration.

was desired in place, without regard to the quality of the air in resisting freezing effects.

When the freezing test results were obtained on the first lots of test specimens, it was felt that something more than total air content of the mix was involved, and that perhaps the quality of the entrained air in the mixture was a factor. It was noted early that the accidental air had a tendency to leave the mixture readily, whereas, with normal aggregates, the entrained air is quite retentive.

The conclusion was that the proper method of handling such an aggregate was to add a large excess of a foaming agent, to increase the air content far beyond any reasonable amount, and add a controlled amount of an anti-foaming agent to reduce the final air content to the amount required in the mixture as discharged from the mixer. Theoretically, in such a procedure, the foaming agent will add stable air to the mixture, and the anti-foaming agent, provided that it is properly selected for the work, will eliminate the more unstable air fractions from the mix, and leave behind only those air fractions which will properly resist freezing effects. This presumes, of course, that the more stable air contents are also those which give greatest resistance to freeze-thaw exposure.

The experience, after a year of tests on concrete from this project, indicates

TABLE 3—CONCRETE OBSERVATIONS (APR. 7, 1952)*

At the mixer			At the forms		
Time	Slump, in.	Air content, percent	Time	Slump, in.	Air content, percent
A.M.			A.M.		
10:05	3	5.0			
10:15	3	5.0	10:15	3	5.5
10:30	2¾	4.9	10:30	3	5.3
10:45	2¾	5.1	10:45	3	5.2
11:00	3	5.6	11:00	3	4.9
11:15	3	4.8	11:15	2½	4.0
11:30	3	5.9			
P.M.			P.M.		
2:15	2½	4.5			
2:30	3	5.5	2:30	3	4.3
2:40	3	5.0			
			2:45	3	4.3
2:55	3	5.5			
			3:00	3	6.4
3:10	3	4.0			
			3:15	3	4.5
3:25	3½	6.4			
			3:30	2½	5.0
3:40	3	5.3			
			3:45	3	5.0
3:55	3	5.0			
			4:00	2½	4.4
4:05	3	5.0			
4:15	3	5.0	4:15	3	4.5
4:30	3	5.5	4:30	3	4.6
4:40	3	5.7			
			4:45	2½	4.1
4:50	3	6.0			
			5:00	3	4.9
5:05	3	5.0			
5:15	3	4.9	5:15	3	5.9
5:30	3	4.9			
Maximum	3½	6.4	Maximum	3	6.4
Minimum	2½	4.0	Minimum	2½	4.0
Average	3	5.2	Average	3	4.9

*Admixtures: air-entraining agent + foam control agent.

that this may be correct. Table 3 presents the results of tests made on Apr. 7, 1952, on concrete delivered to the work.

This presents a much better picture of air control than the record as of June 5, 1951. Of greater importance, perhaps, is the fact that all of the samples of concrete sent in for freezing resistance tests during the period April-May, 1952, some 50 in all, came through 150 cycles of the freeze-thaw test without showing any visual evidence of damage.

SPECIAL INVESTIGATIONS

The observations, both in the laboratory and in the field, indicated that these sands, in addition to the fact that they possessed the property of accidentally entraining air, were extremely susceptible to the addition of further amounts of air-entraining agents. Addition of even a comparatively small amount of any of the normal air-entraining agents would result in an abnormal increase in the air content of the mix.

It was found also that with sands of this type, time of mixing is a pronounced factor in the amount of air entrained. A considerable part of the concrete on these projects was mixed in transit-mix trucks, where mixing time was liable to much greater variation than in central plant mixing operations. It was felt that some further information in regard to these effects would be desirable.

Long mixing time

Some difficulty was experienced at the start of operations in controlling the uniformity of concrete weighed at the batching plant and mixed in transit mixers. At that time the normal procedure was to insure minimum mixing time of 10 min but no maximum limit was specified as the concrete was usually discharged shortly after the completion of the 10-min mixing period. It was found that the air content would increase on longer mixing time and laboratory and field tests were carried out to determine the effect of long mixing time on air content.

Laboratory tests were carried out in a small 2.5-cu ft mixer operating at 22 rpm. The general procedure was to mix the batch until maximum air content was reached and then eliminate the excess air with an anti-foaming agent. Results of tests on two batches of plain concrete are given in Tables 4 and 5.

It appears that the air content in these plain mixtures builds up to a

TABLE 4—MIXING TIME TESTS OF PLAIN CONCRETE (MIX 70h)

Mixing time, min	3	8	13	18	23	28*	30*	33
Air content, percent	6.0	14.0	17.3	20.0	21.0	23.0	12.5	6.4
Unit weight, lb	142.8	130.0	125.2	122.0	120.2	118.6	128.8	146.8
Slump, in.	7	8	8	8	8	8	5	2

*Anti-foaming agent added after 28 and again after 30 min.

TABLE 5—MIXING TIME TESTS OF PLAIN CONCRETE (MIX 70i)

Mixing time, min	3	8	13	18	23	28*	33
Air content, percent	3.8	11.7	13.5	14.4	15.5	15.5	7.2
Unit weight, lb	151.0	136.8	134.2	134.0	132.0	131.2	146.2
Slump, in.	2	7	8	8	8	8	3

*Anti-foaming agent added after 28 min.

practical maximum in 15–20 min of mixing time and, without foam control, in excessive amount. Addition of the foam control agent at the start of the mixing operation reduces total air as well as limiting the effect of excessive mixing time. Table 6 shows the characteristics of a mixture in which the foam control agent was added at the start of the mixing operation.

In the above test it is noted that the air content builds up to a practical maximum after about 20 min of mixing. With further mixing the air begins to reduce in amount, and after 50 min of mixing it is eliminated fairly rapidly. Similar conditions were observed in regard to the concrete discharged from the 2- and 4-cu yd central plant mixers, and it was found necessary, to obtain effective control of air content, to limit mixing time to a maximum as well as a minimum, and to adjust the amount of foam control agent added to that which would give the desired air content after the specified period of mixing.

Similar results were obtained from mixtures prepared in transit-mix trucks, although in these mixers the mixing period required to increase air to a maximum was appreciably greater than in the case of the small laboratory mixer. Results of observations on a batch of plain concrete mixed in a 3-cu yd transit mixer are shown in Table 7.

Here again it was found necessary to limit mixing time to a maximum to avoid excessive air entrainment as a result of long-continued mixing. Mixing time for transit mixers was set at 15 min and rigidly controlled to that time. As in the case of the central mixing plant operations, the amount of foam control agent was adjusted to give the desired amount of air in the concrete on discharge from the mixer.

Effect of foam control agent on concrete

A series of tests was made to determine the effect of the foam control

TABLE 6—MIXING TIME TESTS (MIX 70e)*

Mixing time, min	3	8	13	18	23	53	68
Air content, percent	2.3	3.5	6.2	6.7	6.9	5.6	3.7
Unit weight, lb	152.8	151.0	145.8	143.8	142.6	145.0	150.2
Slump, in.	5	6½	7	7	8	2	½

*Foam control agent added at start of mixing.

TABLE 7—PLAIN CONCRETE MIXED IN 3-CU YD TRANSIT MIXER

Mixing time, min	10	20	40	60	80	100	110	120
Air content, percent	4.0	9.8	12.5	13.4	13.1	11.5	10.1	7.6
Unit weight, lb	151.8	143.2	137.6	136.4	136.6	139.2	141.4	145.2
Slump, in.	1½	6	5	4¾	4¾	2½	2¼	1½
Compressive strength, 28 days, psi	4685	3270	2340	2260	2220	2565	3090	3540

agent on the properties of the concrete, particularly as regards compressive strength and resistance to freezing effects. In carrying out this series the amount of foam control agent was varied from nil to a maximum of 80 lb per cu yd. Mix proportions and results of compressive strength tests are shown in Table 8.

In actual field application it was found that the amount required for satisfactory control of air content varied from 1 to 15 cc per cu yd, so that mix 85b (Table 8) contains somewhat more than twice as much of the foam control agent as was ever used in practice, and mix 85g contains about 2500 times as much. Mix 85b shows somewhat higher strength than mix 85a, probably as a result of the elimination of the greater part of the air. The compressive strengths of mixtures 85c, d, e, and f vary somewhat but the differences are not significant. Mix 85g does, however, show a decided drop in compressive strength, perhaps because that with such a volume of foam control agent in the mixture the hydration of the cement may be impeded.

CONCLUSIONS

1. In the past four or five years, fine aggregates possessing similar characteristics, though generally in lesser degree, have been encountered in numerous locations, among them the following: (1) Sault St. Marie, Ont.; (2) Greenwood, N. S.; (3) La Trenche, Que.; (4) Morin Heights, Que.; (5) Baie Comeau, Que.; (6) Cape Ray, Newfoundland; (7) Chalk River, Ont.; and (8) Sudbury, Ont. All of these aggregates occur in the area covered by the Laurentian shield, and all of them are glacial in origin. Since this paper

TABLE 8—EFFECT OF EXCESS DOSAGE OF FOAM CONTROL AGENT

Mix	85a	85b	85c	85d	85e	85f	85g
Cement, lb	470	471	472	466	469	466	454
Sand, lb	1330	1408	1411	1392	1403	1393	1357
Coarse aggregate, lb	1841	1947	1951	1926	1940	1927	1876
Water, lb	269	298	298	295	291	285	256
Foam control agent	nil	1.25 oz	5.0 oz	1.25 lb	5 lb	20 lb	80 lb
Weight per cu yd, lb	3910	4124	4132	4079	4108	4091	4023
W/C	0.57	0.63	0.63	0.63	0.62	0.61	0.56
Slump, in.	3	2½	2½	2½	2½	2½	3
Air, percent	7.1	1.9	1.8	1.7	1.5	1.6	1.0
Compressive strength, psi							
3 days	1920	2185	2210	2180	2135	2275	1690
7 days	2890	2985	3170	3320	2960	3100	2750
28 days	3750	3830	4050	4295	3830	3975	3525
Degree of scaling	0	1	1	1	2	2	2

was compiled the authors have recorded a number of additional cases where similar conditions have occurred.

This may explain, in part at least, the varying behavior of concrete in some of the structures in our northern areas which have been exposed to winter conditions. A typical example occurs in the case of the concrete works in connection with the power development at Shipshaw, near Arvida, Que. These structures, built 12 years ago without the use of air-entraining agents, are in excellent condition today except for certain localized areas where segregation in placing apparently resulted in concrete of high W/C and poor quality. The great bulk of the work shows no sign of damage as a result of exposure to freezing effects. Tests of the sand from the deposit used as a source of fine aggregate for this concrete indicate that it has the property of occluding air into the mixture to the extent of 3 to 6 percent, and this may account for the excellent performance of this concrete to date.

2. A satisfactory degree of control of air content and consistency can be achieved by adding a surplus of an air-entraining agent to the mix and then eliminating the more unstable air fractions by the addition of a foam control agent in amount sufficient to leave the desired quantity of residual air in the concrete. In actual practice, both the foaming agent and the anti-foaming agent are added at the same time.

3. With sands possessing these characteristics it is necessary to set maximum as well as minimum limits for length of mixing time, both for central mixing and for transit-mix operations.

4. The outlined methods of control of air content in concrete when sands of the type mentioned are encountered yield mixtures which are highly resistant to freezing effects without, as far as has been established, any detriment to the other properties of the concrete.

5. In the majority of cases where this problem is encountered, in the experience of the authors, it would seem that the solution is to obtain aggregates from some alternate source which behave in normal fashion. If material of normal behavior is obtainable within economic distance it should be used. If such material is not available within economic hauling distance, then the information presented above may be of assistance in using the local aggregates to advantage.

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Title No. 51-14

Permeability of Portland Cement Paste*

By T. C. POWERS,† L. E. COPELAND,‡ J. C. HAYES,§ and H. M. MANN‡

SYNOPSIS

Apparatus and methods for measuring the permeability of portland cement pastes are described. Test results are given showing the effects of curing, cement content, cement composition, and cement fineness. Also, data on some rocks are compared with data on hardened pastes.

INTRODUCTION

This paper deals with experiments on the permeability to water of portland cement paste. The relationship of the permeability of the paste to that of concrete as a whole is understood in a general way. The paste is a continuous body enveloping and isolating the individual aggregate particles. The over-all permeability is a function of the paste permeability, the permeability of the aggregate particles, and the relative proportions of the two. Fissures under the aggregate particles formed during the period of bleeding, and cracks caused by volume-change restraint also play a part. The permeability of paste has also an important bearing on the vulnerability of concrete to frost action. It determines the relative ease with which the cement paste and the aggregate may become resaturated after drying, and it is a principal factor determining the destructiveness of freezing—once the paste becomes water-soaked. This latter subject has been treated extensively in other papers.[1,2,3] Studies of paste permeability have thrown light on the question of hydrostatic pressure in the interior of dams. Along with other information they have helped to identify the "ultimate particles" against which hydraulic forces inside the concrete can develop. With these particles identified and their wettable areas measured, the order of magnitude of the area factor for computing hydrostatic uplift forces within concrete dams could be established.[4]

EXPERIMENTAL METHODS

The apparatus used for permeability measurements is shown schematically in Fig. 1, and its actual appearance in Fig. 2. Fig. 1 shows the system in which hydrostatic pressure is produced by standpipes of mercury, and it shows one of the four permeability cells attached to that system.

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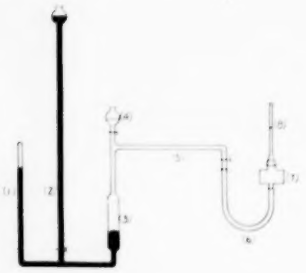

Fig. 1—Schematic drawing of permeability apparatus

(1) Closed End Manometer (5) Pressure Manifold
(2) Hg. Pressure Column (6) Rubber Hose Connection
(3) Hg-H_2O Reservoir (7) Permeability Cell
(4) H_2O Supply Reservoir (8) Capillary Flow Tube

The sample is a truncated cone that fits into the permeability cell in the manner shown in Fig. 3.

As indicated in Fig. 1, the outflow from the sample enters a calibrated capillary that stands vertically. The capillary bore is 1.016 mm, giving a capacity of 8.107×10^{-3} cc per cm. During the course of the test the change in position of the meniscus is measured after various lengths of time by means of a micrometer microscope reading to a micron. A change in the quantity of liquid on the downstream side of the sample as small as one millionth of a cubic centimeter can be measured. To diminish variations in height of the meniscus brought about by variations in temperature, and to keep the viscosity of the flowing water as constant as possible, the permeability cells are immersed in water baths at 27 ± 0.01 C, as shown in Fig. 2. The liquid level in the bath is automatically kept constant by flow from overhead reservoirs, not shown in Fig. 2. Hydrostatic pressure is indicated by a closed manometer, requiring corrections for variations in atmospheric pressure. The water was boiled to eliminate most of the dissolved air before it was put in the pressure chamber.

Preparation of test specimens

Batches of cement-water paste were mixed in a high-speed stirrer of the

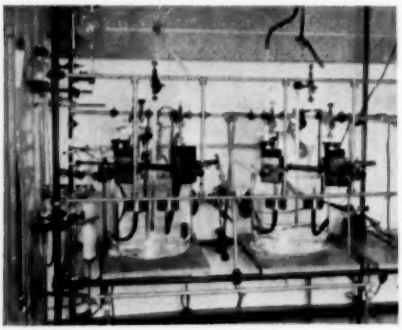

Fig. 2—General view of permeability apparatus

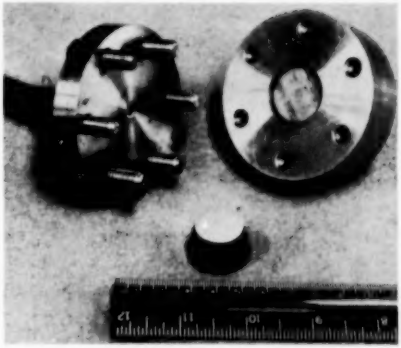

Fig. 3—Close-up of an opened cell with a test piece in place and another test piece, coated with a special grease, illustrating the appearance before the sample is inserted in the cell. A necessary gasket is not shown

Waring Blendor type. In most cases mixing was done under reduced pressure to eliminate the air normally whipped into the paste during mixing. This was done as follows. With the mixer mounted on a pump plate, precooled water was placed in the mixing bowl and the cement in a hopper at the top of the bowl. A bell jar was then set in place, sealed, and the air pumped out by a mechanical vacuum pump operating through a vapor trap. When the pressure dropped to about 25 mm of mercury, an electrical vibrator on the hopper was operated, causing the cement to flow from the hopper into the mixing water. The precooling of the water was such that after a 1-min period of mixing, 3 min of waiting, and then a second 2 min of mixing, the temperature of the paste would be 23 ± 2.0 C (73 ± 4 F).

After mixing, the bell jar was removed and the batch transferred to two 8 x 1⅛-in. test tubes, the top of the paste being about 1.5 in. below the rim of the tube. These molds were stored vertically (at 23 ± 0.5 C; 73 ± 0.9 F) and stoppered to prevent loss of water by evaporation. Before stoppering the tube, extra water was placed on top of the sample, the amount being more than the quantity to be absorbed by the sample during the curing period.

The cylinders of paste were left in the glass molds throughout the curing period. When the time came for a permeability test the samples were obtained as indicated in Fig. 4. Procedure 1 (left side) was used in the early studies; Procedure 2 represents a later improvement.

To obtain the samples the glass is first cut away and then the upper part of the cylinder cut off and discarded. The remaining part was placed in a lathe and tapered. Then slices were cut off as shown. During this operation the sample was kept dripping wet.

The reason for adopting Procedure 2 is indicated in Fig. 5, which shows the measured densities and the corresponding water contents at different levels in hardened paste. At the top the water content is high and the density low—probably because of disturbances produced when placing curing water on top of the fresh paste. (This shows why the top part of the specimen is discarded.) Below this topmost layer the paste may show a zone of constant

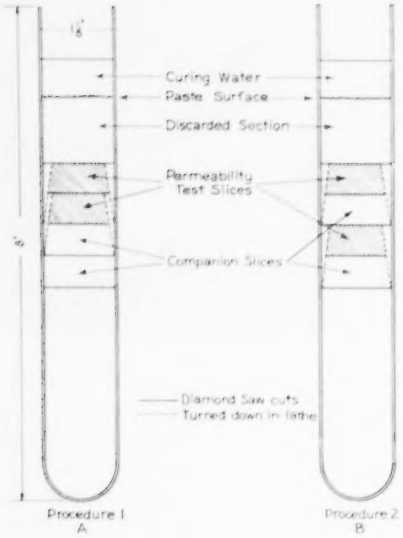

Fig. 4—Relative size and position of test slices with respect to the original specimen

density below which the density increases with distance from the top. These are the normal effects of sedimentation, as discussed in a previous publication.[5]

In a sample such as that shown in Fig. 5, slices cut from the zone of constant density would be comparatively homogeneous and the specimens for permeability tests would have the same composition as those used for auxiliary tests. However, if the slices had been cut from the lower zone, the specimens would not be isotropic and the companion pieces would not be like the permeability pieces.

The pattern shown in Fig. 5 does not represent all samples; the depth of the zone of constant density varies from paste to paste and in some cases the density varies continuously from top to bottom. Procedure 2, as illustrated in Fig. 4, was adopted to reduce the difference between permeability samples and their companions.

In all recent work the auxiliary tests were made on the permeability sample after completion of the permeability test. The auxiliary tests referred to were principally those required to determine the total water content and the nonevaporable water content.*[6] Also, the densities (specific gravities) were measured prior to determining the water contents.

At high water contents, the pastes tend to develop vertical channels during the bleeding period.[5] Such channels probably do not become completely filled with hydration products. The procedure described above did not eliminate this fault. Although the channels were not visible, we believe they might have been present in some specimens and might account for what seems to be abnormally high coefficients of permeability.

Early in these investigations preheating of the cement was used to induce

*The nonevaporable water is practically equal to the chemically combined water.

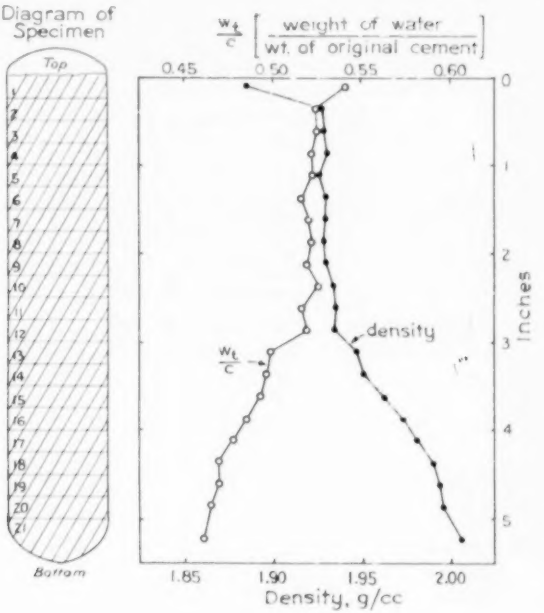

Fig. 5—Total water and density versus distance from top of specimen

false set and thus to produce uniformity of density with depth in a molded specimen. Upon observing that this treatment also resulted in an increased permeability, the preheating was discontinued. With three exceptions the data in this paper were obtained entirely from investigations on specimens prepared without preheating the cement. The exceptions are: (1) Table 2—Effect of Hydration; (2) Table 4—Effect of Cement Fineness; and (3) Table 5—Effect of Drying. We believe that each of these factors would have the same influence, regardless of whether or not the cement had been preheated.

Method of measuring rate of flow

In most tests the samples were placed in the permeability cells, subjected to a constant hydrostatic pressure of about 3 atmospheres, and kept under observation until a steady state of flow was closely approximated. Sometimes the flow rate would become practically constant within 3 or 4 days, but often the required period was as long as 4 weeks. During the period of observation the rate of flow was measured at least once a day by making four or five readings.

Calculation of coefficient of permeability

The fundamental definition of the coefficient of permeability may be stated as follows, using Muskat's development[7] and the nomenclature of a previous paper.[8]

$$\frac{dq}{dt} \times \frac{1}{A} = \frac{K_2}{\eta} \times \frac{\Delta P}{L} \qquad (1)$$

where dq/dt = rate of flow in cu cm per sec
A = mean cross-sectional area of sample, sq cm
η = viscosity of water at the temperature of the experiment
ΔP = pressure drop across specimen, dynes per sq cm
L = thickness of sample, cm
K_2 = coefficient of permeability, sq cm

By this definition the coefficient of permeability is a property of the paste alone; theoretically, the same result would be obtained with different liquids by using their respective coefficients of viscosity η. However, for flow of water through cement paste we have reason to believe that the coefficient of viscosity is not a constant. When water flows through channels as small as those in cement paste, the viscosity appears to be a function of the size of the channel. Therefore, for our present purpose, it is advantageous to use the following definition of permeability coefficient:

$$\frac{dq}{dt} \times \frac{1}{A} = K_1 \frac{\Delta h}{L} \quad \ldots\ldots (2)$$

Here Δh = drop in hydraulic head across specimen and K_1 is in cm per sec. It is related to K_2 as follows:

$$K_1 = \frac{K_2 d_f g}{\eta_e}$$

where d_f = density of the fluid, g per cu cm
g = acceleration due to gravity, cm per sec per sec
η_e = effective viscosity, poises

It thus embodies the properties of the solid and the liquid, whatever may be the manner and degree to which their properties may be altered by physical interaction. When K_2 is calculated on the basis of $\eta_e = 0.0085$ poise,

$$K_1 = 1.15 \times 10^5 K_2 \quad \ldots\ldots (3)$$

Neither Eq. (1) nor Eq. (2) applies exactly to flow through a truncated cone, which is the shape of these test specimens. An exact equation for the flow through a body so shaped cannot be obtained. We used Eq. (2) with A calculated from

$$A = \frac{\pi}{8} (d_1^2 + d_2^2) \quad \ldots\ldots (4)$$

where A is the mean cross-sectional area and d_1 and d_2 are the two end diameters of the truncated cone.

Correction for osmotic pressure

During an experiment different alkali concentrations develop in the different parts of the permeability cell. The concentration on the high-pressure side becomes different from that on the low-pressure side and both these concentrations may be different from the average concentration in the specimen. This gives rise to osmotic pressure. In recent work the permeability coefficient was calculated from the net pressure. That is, for Eq. (2),

$$\Delta h = \Delta h_a + \Delta h_o$$

where Δh_a = applied hyrdaulic head
Δh_o = hydraulic head due to osmosis

The head due to osmosis could be either positive or negative.

Most of the data reported in this paper are not corrected for osmotic pressure. The indications are that corrections would be small in most cases —not over 10 percent.

TEST RESULTS

Permeability of fresh paste

As shown by Steinour,[9] fresh paste has structure and can be treated as a porous solid even before the cement sets. Under the force of gravity, the particles in cement paste settle, the rate of settlement being proportional to the permeability of the mass. It can be shown that the coefficient of permeability is directly proportional to the rate of bleeding. For the materials used in these tests, the proportionality constant is 1/2.15.

The permeabilities of fresh pastes as calculated from measured bleeding rates are given in Table 1 for four different cements and one water-cement ratio. The bleeding rate data were published several years ago.[10]

These data indicate the order of magnitude of the permeabilities at a given water-cement ratio. The figures are to be compared later on with those for hardened paste. They show also that when the specific surfaces of the cements and the water-cement ratios of the pastes are equal, differences in permeability of fresh paste are not large, even though the cements differ considerably in chemical analysis.

Permeability of hardened paste

Effect of cement hydration—The chemical reactions between the constituents of portland cement and water progressively replace the original cement minerals with hydration products, principally cement gel. The volume of the cement gel (including gel pores) produced by hydrating the cement is approximately 2.3 times the volume of the cement. Consequently, the gel not only replaces the original cement minerals but also tends to fill the originally water-filled space. Table 2 shows the effects of these internal changes on the coefficient of permeability. The data pertain to a given paste at different stages of its hydration. Notice that within a week the coefficient of permeability dropped to about one one-hundred-thousandth of its initial value. By the 24th day it had dropped to less than a millionth of its initial value.

TABLE 1—PERMEABILITY OF FRESH PASTES

Cement* No.	Specific surface (Wagner)	W/C by weight	$K_1 \times 10^8$, cm per sec
15754	1800	0.5	56
15756	1800	0.5	63
15758	1800	0.5	51
15763	1800	0.5	84

*See Table 3 for compositions.

TABLE 2—REDUCTION OF PERMEABILITY BY CEMENT HYDRATION*

Age	Permeability coefficient K_1, cm per sec
fresh	2×10^{-4}
5 days	4×10^{-8}
6 days	1×10^{-8}
8 days	4×10^{-9}
13 days	5×10^{-10}
24 days	1×10^{-10}
ultimate	0.6×10^{-10} (calculated)

*Cement No. 15754; specific surface = 1800 (Wagner); W/C = 0.7 by weight.

TABLE 3—REDUCTION OF PERMEABILITY BY CEMENT HYDRATION

Cement No.	Specific surface (Wagner)	W/C	K_1, cm per sec	
			Fresh	Ultimate*
15754	1800	0.5	5.6×10^{-5}	4.4×10^{-12}
15756	1800	0.5	6.3×10^{-4}	5.2×10^{-12}
15758	1800	0.5	5.1×10^{-4}	5.7×10^{-12}
15763	1800	0.5	8.4×10^{-5}	6.3×10^{-12}

Computed potential compound compositions

Cement	C_3S	C_2S	C_3A	C_4AF
15754	45.03	25.80	13.34	6.69
15756	48.51	27.90	4.63	12.60
15758	60.57	11.58	10.26	7.76
15763	28.33	57.52	2.22	5.96

*Approximate, by calculation.

Table 3 shows the total change in permeability for comparable pastes made with four different cements. The data in the fourth column are those given in Table 1. The fifth column gives approximately what the permeabilities would be after all the cement in the paste had become hydrated.

Effect of varying the cement content—The permeability of cement paste at a time when a given percentage of the cement has become hydrated is lower the higher the cement content of the paste. The relationship for a series of pastes in which about 93 percent of the cement was hydrated is given in Fig. 6A.

Fig. 6B gives data for specimens prepared from the same cement as in Fig. 6A, but for the specimens in Fig. 6B the cement had been treated in such a way as to induce false set. The cement was heated over night at 105 C so as to reduce the gypsum to the hemihydrate. The two curves show that at a given water-cement ratio (and at approximately the same

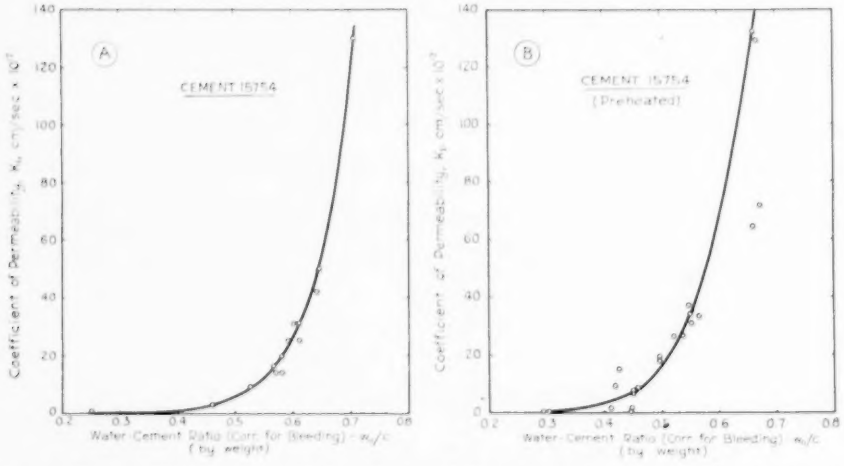

Fig. 6—Relationships between coefficient of permeability and water-cement ratio for mature paste

percentage of cement hydration) the coefficients for specimens made with the preheated cement were about double those for the other specimens.

We do not know whether or not there is any connection between the existence of false set and the slight increase in permeability. All we know is that the treatment that produced false set also increased the permeability coefficient.

Effect of chemical composition of the cement—In Table 3 the initial and final coefficients of permeability are shown for four cements having different chemical compositions. The figures for the ultimate coefficients are not direct test results but are values calculated from general relationships. The data are for pastes of one water-cement ratio; comparisons at other water-cement ratios give similar indications.

Influence of specific surface of cement—The cements used in this investigation were available at different degrees of fineness, the different grinds being made from the same clinker in a commercial plant. The permeabilities of pastes made with these cements are given in the fifth column of Table 4.

The figures in the last column, taken from the smooth curve drawn in Fig. 6B, may be used for comparison with the permeabilities of pastes made with cements of different fineness, and, with one exception, different chemical composition. The exception is the first item, cement No. 15364, which was made from the same clinker as cement No. 15754 represented in the last column.

TABLE 4—EFFECT OF DIFFERENCE IN CEMENT FINENESS ON PERMEABILITY OF HARDENED PASTE*

Cement lot No.	Specific surface (Wagner)	W_o/C by weight	Percent hydration	Permeability $K_1 \times 10^{13}$, cm per sec	Cement No.15754; specific surface = 1800; percent hydration = 93; $K_1 \times 10^{13}$, cm per sec
15364	1040	0.54	79	25	
		0.52	79	20	
		0.56	74	32	
		0.55	74	29	
		0.54	76	26	30
15622	2200	0.61	72	189	
		0.61	72	132	
		0.61	72	125	
		0.61	72	147	
		0.61	72	148	75
15495	1470	0.57	79	34	
		0.56	79	40	
		0.56	79	37	42
15496	1740	0.60	88	34	
		0.60	88	32	
		0.61	86	28	
		0.61	86	38	
		0.60	87	33	67
15497	2500	0.61	91	25	
		0.61	91	30	
		0.61	91	28	75

*Cements preheated to reduce bleeding.

These data show that pastes having the same water-cement ratios are likely to have similar though not identical degrees of permeability after the cements have reached fairly advanced stages of hydration. The last three cements listed were all made from the same clinker and thus differed only in the degree of fineness. Even though the highest specific surface is nearly double the lowest, the coefficient of permeability of the finest is only about 25 percent less than that of the coarsest.

The first cement listed, No. 15364, may be compared directly with No. 15754 (sixth column). Here, although the degrees of hydration and the specific surfaces are considerably different, the coefficients of permeability are almost identical.

The pastes made with cement No. 15622, a Type II composition, appear to be somewhat more permeable than those made from the other cements. However, the difference may disappear at a later age, when the degrees of hydration would be more nearly equal. In general, the data indicate that a paste made from a coarsely ground cement may be just as impermeable as one made from a finer cement.

Effect of drying—All the data reported above are from tests on specimens kept wet continuously. To determine the effect of drying, two pairs of tests were made on the pastes that were partially dried after the curing period. One paste had been cured in the glass mold for 141 days and the other for 63 days. The pastes were then stored in closed glass vessels as follows:

First drying stage—208 days at 93 percent relative humidity
Second drying stage—1040 days at 79 percent relative humidity
Third stage—238 days at 100 percent relative humidity

Samples were shaped for permeability tests at the end of the second drying stage. During the two drying stages the specimens reached equilibrium with 79 percent humidity. In the third stage they returned to about 97 percent of saturation.*

By this procedure the moisture changes occurred gradually, and the stresses arising from differential shrinking or swelling were correspondingly low. Preliminary tests showed that drying a specimen rapidly by exposing it to a low humidity would crack it, or placing a dry specimen in water to resaturate it would produce stresses high enough to destroy the specimen.

TABLE 5—PERMEABILITY OF DRIED PASTES†

Ref. No.	W/C	Porosity of paste‡	Coefficient of permeability $\times 10^{12}$, cm per sec
L-4-A-A	0.50	0.410	990
L-4-A-B	0.50	0.412	915
L-27-A-A	0.50	0.408	400
L-27-A-B	0.50	0.404	1025

†Cement No. 15754 (preheated).
‡At time of test.

The permeabilities and porosities of two samples of each paste are given in Table 5. Three out of four specimens show permeability coefficients in the neighborhood of 1000×10^{-12} cm per sec. Other tests on comparable specimens (same porosities, same cement), but not subjected to drying and resaturating, gave coefficients

*The schedule used represented much more than the shortest periods needed for each stage.

TABLE 6—APPARENT EFFECT OF ALKALI CONTENT ON PERMEABILITY*

		Permeability K_1, cm per sec, at		
		3 days	7 days	14 days
Leached	A	24.2×10^{-12}	24.6×10^{-12}	23.8×10^{-12}
	B	18.9×10^{-12}	20.0×10^{-12}	21.7×10^{-12}
Not leached	A	13.5×10^{-12}	15.5×10^{-12}	19.3×10^{-12}
	B	13.1×10^{-12}	14.5×10^{-12}	17.4×10^{-12}

*Cement No. 15761 — Na_2O = 1.13 percent; K_2O = 0.44 percent.

close to 15×10^{-12}. Thus, gradual drying to 79 percent relative humidity increased the coefficients of permeability about seventy-fold.

It is likely that the effect of drying on the coefficient of permeability would be greater than that indicated above if the specimens had been dried to equilibrium with a lower humidity. Specimens dried to various degrees are available but have not yet been tested.

From other data, we believe that the capillary space (as distinguished from gel pores) is in the form of isolated cavities, each cavity being surrounded by gel. We believe that shrinkage produced by drying may rupture some of the webs of gel between capillary cavities and thus increase the permeability. We found no evidence microscopically of cracks in the specimens.

Apparent effect of alkali content—Two sets of specimens from samples made with a high-alkali cement were prepared. One set was stored in limewater long enough to leach out most of its alkali, while the other set was kept in a saturated atmosphere. After the alkali had been removed from the one set, both sets were tested for permeability. The results are given in Table 6.

The leached specimens showed little or no change in permeability coefficient between the 3rd and 14th days that they were under test, whereas the unleached specimens showed a nearly 50 percent increase in coefficient of permeability. This might seem to indicate that when alkali is present in a specimen it lowers its permeability and that as the alkali is removed the permeability is increased. However, the indications are that after 3 days under test the alkali concentration in the water on the low-pressure side of the specimen was lower than it was in the water on the high-pressure side. Consequently, osmotic pressure counter to the applied pressure existed and reduced the rate of flow. As time went on, flow of water and diffusion of alkali tended to equalize the concentrations and thus reduce the osmotic pressure. As a consequence, more of the applied pressure became effective and the permeability coefficient apparently increased. Thus, these data do not indicate whether or not the alkali content of cement has any effect on the permeability of the paste made with that cement. The indications are that whatever the effect may be, it is small.

Comparison of hardened paste with rocks

Data on the permeabilities of various rocks are given in the first four columns of Table 7. Conical test samples were cut from selected pieces of

TABLE 7—PERMEABILITY OF ROCKS COMPARED WITH THAT OF HARDENED PASTE

Sample No.	Density, g per cc	Evaporable water capacity, g per cc of sample	K_1, cm per sec	Mature paste of same permeability, W/C		Evaporable water capacity, g per cc of paste
				by weight	gal. per sack	
4	2.99	0.0057	3.45×10^{-13}	0.38	4.3	0.30
6	2.70	0.0082	9.20×10^{-13}	0.41	4.6	0.34
2	2.94	0.0065	1.15×10^{-12}	0.42	4.7	0.35
3	2.65	0.0018	1.26×10^{-12}	0.42	4.7	0.35
9	2.71	0.0046	1.72×10^{-12}	0.44	5.0	0.36
11	2.78	0.0180	3.34×10^{-12}	0.48	5.4	0.39
12	2.75	0.0310	8.05×10^{-11}	0.66	7.4	0.510
5	2.60	0.0140	1.15×10^{-10}	0.68	7.7	0.515
1	2.72	0.0510	2.30×10^{-10}	0.69	7.8	0.522
8	2.69	0.0073	7.48×10^{-10}	0.70	7.9	0.529
7	2.58	0.0430	1.72×10^{-9}	0.71	8.0	0.531
10	2.60	0.0052	2.18×10^{-9}	0.71	8.0	0.532

rock, tested for permeability, and then for total capacity for evaporable water.

The data are divided into two groups: (1) Rocks showing a coefficient of permeability less than 10^{-11} cm per sec. The capacity for evaporable water of all but one of these was less than 1 percent. (2) Rocks having coefficients of permeability greater than 10^{-11} cm per sec. In this group the capacities for evaporable water range from 0.5 to 5.0 percent. The fifth and sixth columns give the water-cement ratios of mature pastes that would have the same coefficients of permeability as the corresponding rock samples. The last column gives the capacities of the pastes for evaporable water. These comparisons show that the rock having the lowest degree of permeability was comparable with a mature cement paste of $W/C = 0.38$ by weight (4.3 gal. of water per sack of cement). For the rock of highest permeability the water-cement ratio of a comparable paste is about 0.71 (8 gal. of water per sack of cement).

Table 7A gives the source and description of the rock samples. The samples probably represented the rocks having minimum permeabilities, for they were selected pieces free from seams or visible imperfections. Data assembled by Ruettgers, Vidal, and Wing[11] indicate that when samples larger than ours were tested, permeability coefficients ranging several orders of magnitude higher than the highest shown in Table 7 were found.

TABLE 7A—DESCRIPTION OF SAMPLES

Sample No.	Lot No.	Source	Description
4	18278	Eau Claire, Wis.	Trap rock, dense, some crystal-boundary pores
6	18393A	C. H. Scholer	Marble, fine grained, dense
2	—	Phillips, Wis.	Quartz diorite, coarsely crystalline, crystal-boundary pores
3	18278	Eau Claire, Wis.	Quartz-feldspar, felsite, very dense
9	—	Thomaston, Me.	Limestone, crystalline
11	—	Elmhurst, Ill.	Limestone, crystalline; fine-grained marble
12	—	Elmhurst, Ill.	Limestone, crystalline; fine-grained marble
5	18278	Eau Claire, Wis.	Quartzite, imperfectly cemented; sandstone
1	—	Elgin, Ill.	Limestone, uniform, fairly dense, pure
8	18460	Santeetlah Dam	Granite, gray
7	18462	Raleigh County, W. Va.	Sandstone, porous
10	18200	Lithonia, Ga.	Granite

These data show that the pore size of a typical rock is much larger than the pore size of a comparable hardened cement paste. A rock having an evaporable water capacity of less than 0.005 g per cc may have a permeability coefficient equal to that of a paste having a water capacity of 0.35 g per cc.

SUMMARY OF TEST RESULTS

(1) The permeability coefficients of fresh paste, $W/C = 0.5$, range from 5×10^{-5} to 8×10^{-5} cm per sec for four cements having different chemical analyses but the same specific surface, 1800 Wagner. The permeability coefficient for $W/C = 0.7$ was 2×10^{-4} for the same cement that gave 0.6×10^{-4} at $W/C = 0.5$.

(2) The permeability of mature, hardened paste is between 1 millionth and one 10-millionth of that of fresh paste. It ranges from 0.1×10^{-12} to about 120×10^{-12} cm per sec for water-cement ratios ranging from 0.3 to 0.7 by weight.

(3) Mature, hardened pastes made with coarse-ground cements are no more permeable than those made with fine-ground cements when the pastes have equal total porosities. The indications are that they are slightly less permeable. However, the ultimate porosities of pastes made with coarse-ground cements are likely to be higher than those made with fine-ground cements if the initial water-cement ratios (corrected for bleeding) are equal.

(4) Pastes made with portland cements differing in chemical composition have similar permeabilities when the initial water-cement ratios (corrected for bleeding) are equal and when equal fractions of the different cements have become hydrated. At a given age and given water-cement ratio, pastes made with cements that hydrate slowly will have higher coefficients of permeability than those made with cements that hydrate rapidly.

(5) The foregoing conclusions pertain to the permeabilities of pastes that have never been allowed to dry. Drying increases the permeability. For the particular specimens reported here, drying at 79 percent relative humidity increased the permeability about seventy-fold.

(6) Samples of various rocks free from visible flaws had permeability coefficients ranging from 3×10^{-12} to 2×10^{-9}. This corresponds to the permeability coefficient of mature, hardened paste having a water-cement ratio 0.38 by weight at the low extreme and 0.7 by weight at the high extreme (4.3 and 8.0 gal. per sack, respectively).

ACKNOWLEDGMENT

We are indebted to Dr. L. S. Brown for the descriptions of rock samples given in Table 7A and to George Verbeck for selecting and shaping the rock samples used for permeability tests.

REFERENCES

1. Powers, T. C., "A Working Hypothesis for Further Studies of Frost Resistance of

Concrete," ACI JOURNAL, Feb. 1945, *Proc.* V. 41, pp. 245-272; Portland Cement Assn. *Bulletin* No. 5.

2. Powers, T. C., "The Air Requirement of Frost-Resistant Concrete," *Proceedings*, Highway Research Board, V. 29, 1949, p. 184; Portland Cement Assn. *Bulletin* No. 33.

3. Powers, T. C., and Helmuth, R. A., "Theory of Volume Changes in Hardened Portland Cement Paste During Freezing," *Proceedings*, Highway Research Board, V. 32, 1953, p. 285; Portland Cement Assn. *Bulletin* No. 46.

4. Powers, T. C., "Hydrostatic Pressure in Concrete," submitted to American Society of Civil Engineers.

5. Powers, T. C., "The Bleeding of Portland Cement Paste, Mortar and Concrete, Treated as a Special Case of Sedimentation," Portland Cement Assn. *Bulletin*, No. 2, 1939.

6. Copeland, L. E., and Hayes, J. C., "Determination of Nonevaporable Water in Hardened Portland Cement Paste," *ASTM Bulletin*, No. 194, Dec. 1953, p. 70; Portland Cement Assn. *Bulletin* No. 47.

7. Muskat, M., *The Flow of Homogeneous Fluids Through Porous Media*, McGraw-Hill Book Co., New York, N. Y., 1937.

8. Powers, T. C., and Brownyard, T. L., "Studies of the Physical Properties of Hardened Portland Cement Paste. Part 7—Permeability and Absorptivity," ACI JOURNAL, Mar. 1947, *Proc.* V. 43, p. 865; Portland Cement Assn. *Bulletin* No. 22, Part 7.

9. Steinour, H. H., "Rate of Sedimentation: Suspensions of Uniform-Size Angular Particles," *Industrial & Engineering Chemistry*, V. 36, Oct. 1944, p. 901.

10. Steinour, H. H., "Further Studies of the Bleeding of Portland ment Paste," Portland Cement Assn., *Bulletin* No. 4, 1945.

11. Ruettgers, A., Vidal, E. N., and Wing, S. P., "An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam," ACI JOURNAL, Mar.-Apr. 1935, *Proc.* V. 31, pp. 382-416.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Bridges

Hollow precast concrete units of great size form bridge substructure

BEN C. GERWICK, JR., *Civil Engineering*, V. 24, No. 4, Apr. 1954, pp. 59-63

Describes the placing of the hollow precast piers for the Richmond-San Rafael Bridge in California. The piers were placed in sections on precast base grids. The grids were anchored with H-piles, some as long as 195 ft.

Each pier consisted of two circular grids, in which were cast holes to receive the piles, tied together with a spreader truss. Next came two precast, cylindrical bottom shells, each weighing 80 tons. A 5-ft lift of tremie concrete was placed in these shells. On top of the bottom shells was placed a single precast unit consisting of two hollow truncated cones joined by a diaphragm with a total weight of 130 tons. A hollow precast shaft was placed on each cone and a cast-in-place diaphragm connected the two. The entire hollow section was filled with tremie concrete to 5 ft above the water surface.

Experimental investigation of the quality of concrete used for the reconstruction of the Villeneuve-Saint-Georges bridge on the Seine (Etude experimentale de la qualite du beton mis en oeuvre pour la reconstruction du pont de Villeneuve-Saint-Georges sur la Seine)

MICHEL BONNET, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 2, Mar.-Apr. 1953, pp. 185-228; No. 3, May-June 1953, pp. 287-314

Reviewed by ARON L. MIRSKY

Since the Villeneuve-St. Georges bridge (*Annales des Ponts et Chaussees*, V. 123, No. 1, Jan.-Feb. 1953, pp. 9-48; "Current Reviews," ACI JOURNAL, June 1953, *Proc.* V. 49, p. 978) was regarded as a prototype bridge, a comprehensive program of field tests and laboratory investigations was undertaken, and correlation of the results was attempted. Prisms, cubes, and cylinders were tested. Statistical analysis leads to author's proposal of a criterion for determining quality of concrete in place from control specimens taken under rigidly controlled conditions during placing operations, with the variability (scatter) of test values given due consideration.

New bridge forms prove economical

Contractors and Engineers, V. 50, No. 12, Dec. 1953, pp. 50-54

Describes the construction of three reinforced concrete highway bridges in Texas with a total length of 1440 ft, using a new standard bridge form design, for less than $250,000.

Model tests to determine the influence of torsional stiffness of a slab and girder bridge (Modellversuche ueber den Einfluss der Torsionssteifigkeit bei einer Plattenbalkenbruecke)

G. MARTEN, *Bulletin* No. 11, Deutscher Ausschuss fuer Stahlbeton, Berlin, 1952, 26 pp.

Reviewed by RUDOLPH FISCHL

A scale model of a proposed reinforced concrete bridge on three supports, with secondary girders in midspan, was cast by using Silumin, an aluminum alloy. The measurements taken on this homogeneous and isotropic model were compared with the com-

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 3, Nov. 1954, *Proceedings* V. 51. Address 18263 W. McNichols Rd., Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parenthesis following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

puted results of a similar system which was free of torsional stresses.

Deflections and elongations were measured and the bending stresses determined. The effective flange width proved to be a multiple of the value given by the German specifications.

The flexural stresses of the model, compared with those found in the comparative torsion-free design, showed the following results: secondary girders 53 to 76 percent, marginal girders 96 to 120 percent and interior girders 67 percent. Torsional stresses in the marginal girder were 53 percent larger than the shear stresses, and 4.3 times as large as the shear stresses for the interior girders.

Buoyant-type bases support bridge piers

Contractors and Engineers, V. 51, No. 1, Jan. 1954, pp. 62-63

Reinforced concrete caissons were used as floating bases for New York's Tappan Zee thruway bridge. The caissons were constructed in a large natural drydock about 10 miles upriver and floated to the site. They are anchored by 30-in. piles which carry live load and 20 percent of dead load. Buoyancy of caissons supports 80 percent of dead load.

Reinforced concrete arch bridge in Brazil

E. Harold Sidwell, *Concrete and Constructional Engineering* (London), V. 49, No. 6, June 1954, pp. 181-186

Describes the reinforced concrete arch bridge across the Rio das Antas in southern Brazil. This is believed to be the second largest arch span in the world. An appendix gives codes used to determine loads.

Dams

Dams and control works

U.S. Bureau of Reclamation, 3rd Edition, 1954, 218pp., $2.75 (available from Superintendent of Documents, U. S. Government Printing Office, Washington, D. C.)

Presents an outline and summarization of the Bureau of Reclamation's experience in the design and construction of dams and control works. Articles on individual structures exhibit the wide range of sizes, types, and designs of dams, spillways, and outlet works that the Bureau has used under varied topographic, foundation and climatic conditions, materials availability, and water need and use.

In contrast with earlier editions, the presentation has been simplified to widen the range of interest. Detailed design drawings and site layouts have been replaced by simplified, functional representations. A special article on the over-all phases of dam design covers general considerations and problems; another article describes types and usage of gates and valves. Separate articles cover 22 different dam projects.

Model investigations for the Lumiei arch dam (Die Modellversuche fuer die Bogenstaumauer Lumiei)

W. Herberg, *Der Bauingenieur* (Berlin), V. 28, No. 11, Nov. 1953, pp. 402-407

Reviewed by Aron L. Mirsky

Interesting summary of the methods used and the results obtained in investigating this Societa Adriatica di Elettricita dam, which has a total height of 131.85 m (432.6 ft) and a maximum thickness of 13.68 m (44.88 ft).

Some recent Swiss hydroelectric schemes

The Engineer (London), V. 197, 1954; No. 5124, Apr. 9, pp. 518-520; No. 5125, Apr. 16, pp. 554-555; No. 5126, Apr. 23, pp. 591-593; No. 5129, May 14, pp. 698-701; No. 5130, May 21, pp. 734-736; No. 5131, May 28, pp. 770-772; No. 5132, June 4, pp. 805-807; No. 5133, June 11, pp. 842-844; No. 5134, June 18, pp. 878-882; No. 5135, June 25, pp. 914-918

Reviewed by Aron L. Mirsky

Extensive and interesting survey of Swiss hydroelectric developments, including the Mauvoisin, Grande Dixence, Marmorera, Wildegg-Brugg, Birsfelden, Aare River (Oberhausli Co.), and Maggia River schemes. Coverage includes dam and power plant design and construction.

New hyrdoelectric dams to spur power program

Albert C. Smith, *Contractors and Engineers*, V. 51, No. 6, June 1954, pp. 26-31

Describes construction of Indian Pond Dam on the Kennebec River, Me. An interesting feature of the job is the tautline cableway used in placing the concrete. The 10-ton capacity cableway consists of a 125-ft fixed head tower and a 65-ft movable tail tower with a maximum span of 900 ft.

Design

Distribution of end reactions of concrete floor slabs under concentrated loads

M. G. Spangler, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 144-160

Author's Summary

This paper deals with the problem of the lateral distribution of the end reaction, or end shear, of a reinforced concrete floor slab resting on two relatively rigid supports, when the

slab is subjected to concentrated loads such as the wheels of a truck.

Bulletin No. 126 of the Iowa Engineering Experiment Station, published in 1936, reported experimental results on 20 individual slab specimens. Results indicated the thickness of the slab is the most important factor which influences the effective width to be used in the computation of vertical end-shearing stresses in this type of structure. To extend the range of thicknesses of the experimental slabs, one additional slab has been constructed and tested.

Data obtained with this new, thicker slab supplement those previously reported in *Bulletin* No. 126, which are included in the paper. Conclusions are drawn on the basis of results obtained on all 21 of the experimental slabs. The principal conclusion is embodied in an empirical formula for effective width for a single concentrated load applied adjacent to a support and in the vicinity of the longitudinal centerline, which is $e = 6\sqrt{t}$ in which e is the effective width and t is the thickness of slab, both in feet. Modifications of this formula for loads placed near a free edge and for multiple loads are given.

Columns with discontinuity in moment of inertia and axial load (Pelare med diskontinuitet i troghetsmoment och axialbelastning)

J. M. Monsted, *Betong* (Stockholm), V. 38, No. 2, 1953, pp. 119-125

Reviewed by Eivind Hognestad

Buckling loads are studied for simply-supported and fixed-end columns axially loaded with a force at the top and another force acting below the top. The cross section changes at the height where the lower force acts. Solutions are given in tables.

Use of orthogonal functions in solving problems of the theory of elasticity. V. 1—General: Torsion (in Portuguese)

Telemaco van Langendonck, Associacao Brazileira de Cimento Portland, Sao Paulo, Brazil, 1952

Reviewed by J. Laginha Serafim

Basic notions on orthogonal functions are reviewed with a view to their application to solving problems formulated by means of Poisson's and Airy's differential equations.

The present volume presents the resolution of the problem of prismatic bars subject to torsion, the solution being obtained for particular cases of the corresponding Poisson's equations. The following cross section shapes are dealt with: lozenge, cross, T, L, and segment of circle. The author further advances a method based on the same analytical tools, for determining the polar moment of inertia of such hollow sections as a square with a square hole, a square with a round hole, and a circle with a square hole.

As a further application of the orthogonal functions an investigation is presented on the torsion of revolution bars of variable cross section.

This material was partly published by the author (in French) in *Preliminary Publications*, Fourth Congress, International Assn. for Bridge and Structural Engineering, Cambridge and London, 1952, p. 189.

Time factor in the mechanical straining of rigid materials (Der Zeiteinfluss bei der mechanischen Beanspruchung fester Stoffe)

E. Walloschke, *Der Bauingenieur* (Berlin), V. 28, No. 12, Dec. 1953, p. 451

Reviewed by Aron L. Mirsky

Discussing paper by Helmut Koerner (*Der Bauingenieur*, V. 28, No. 1, Jan. 1953, pp. 12-17; "Current Reviews," ACI Journal, June 1953, *Proc.* V. 49, p. 969), author demonstrates that Maxwell's relaxation theory is not exactly applicable to concrete.

Reinforcement of slabs and shells designed according to the theory of elasticity (Armering av elasticitetsteoretiskt beraknade plattor, skivor och skal)

A. Hillerborg, *Betong* (Stockholm), V. 38, No. 2, 1953, pp. 101-109

Reviewed by Eivind Hognestad

Theoretical discussion of the amount of reinforcement necessary to carry bending as well as twisting moments in slabs or normal and shearing forces in shells. Only orthogonal reinforcement is considered.

Practical considerations in the design and construction of concrete structures for waterworks

C. V. Green, *Journal of the Institute of Water Engineers*, V. 7, No. 6, Oct. 1953, pp. 465-496

Reviewed by M. W. Jackson

Reviews the basic requirements of concrete and concrete construction methods in reservoirs and similar structures.

Beginning with the water-cement ratio, "more concrete is probably spoiled by the incorrect use of water in the mix than by any other single cause." Aggregates, cement, mixing of concrete, handling of concrete, placing of the concrete, use of admixtures, and concreting under special conditions are

covered. Additional tests to measure imperviousness are described.

Prevention of cracking depends on low concrete tensile stresses and proper expansion and contraction joints. Suggested optimum spacing of complete contraction joints is 25 ft, though some discussers disagreed.

Stress distribution between reinforcement and concrete when the stresses are regarded as plane (Spanningsfordelning mellan armering och betong vid plant spanningstillstand)

S. E. Andersson, *Betong* (Stockholm), V. 38, No. 2, 1953, pp. 111-118

Reviewed by Eivind Hognestad

Stresses in a concrete body, in which several reinforcing bars are embedded side by side, are studied. Cross section of the reinforcement is approximated by a rectangle with a width equal that of the concrete body. Thereby a plane stress distribution results, and an elastic analysis is possible with the aid of Airy's function.

Design calculations for cylindrical shell roofs

J. E. Gibson, *Civil Engineering and Public Works Review* (London), V. 48, No. 568, Oct. 1953, pp. 943-945; No. 569, Nov. 1953, pp. 1055-1057; No. 570, Dec. 1953, pp. 1157-1159; V. 49, No. 571, Jan. 1954, pp. 72-74

Reviewed by James Michalos

Presents approximate methods for the analysis of short and long cylindrical shells. Concludes that well known membrane theory is adequately accurate for analysis of short shells. Long shells are analyzed by making use of Schorer's approximation. Tables are given to aid in the solution, and numerical examples are included.

Strength of materials—theoretical and experimental. V. 1. Elasticity theory for structures (in French)

R. L'Hermite, Editions Dunod, Paris, 1954, 860 pp., 8400 Fr.

Applied Mechanics Review May 1954 (Moorman)

Subjects covered are: theory of elasticity; plane elasticity; internal equilibrium of bodies, one of whose dimensions is great with respect to the other two (*e.g.*, beams); tension and flexure; torsion; theory of statically determinate structures; statically determinate straight, curved, and hinged members; theory of statically indeterminate structures for beams and systems of straight members, also for curved beams, arches, rigid frames, and three-dimensional structures; plates and slabs in flexure; thin shells, including prismatic plate structures. Very thorough treatment is given in the fields of theory of elasticity, advanced mechanics of materials, and of plates and shells. In the field of statically indeterminate structures a general survey is made and then complete development is given for topics such as horizontally curved beams and beams on elastic supports. Notation is listed at the end of each chapter. More than 700 references are listed at the ends of chapters in addition to many more footnotes.

Reviewer believes book to be the best documented piece of work in its field and should be accessible to all those engaged in machine design, structural and aeronautical stress analysis, and in the field of theoretical and applied mechanics.

Marginal cost in engineering design

D. F. Coates, *The Engineering Journal* (Montreal), V. 37, No. 3, Mar. 1954, pp. 246-248

Reviewed by Aron L. Mirsky

A concise presentation of a topic familiar to too few engineers, but one likely to grow in importance as the world becomes more cost-conscious. As illustration, author investigates the economic height of a concrete dam for a hydroelectric project.

Materials

Pozzolanas—II. Behavior of pozzolana-lime mortars of low water-cement ratio with particular reference to the problem of assessing pozzolanic reactivity

K. M. Alexander, *Australian Journal of Applied Science* (Melbourne), V. 5, No. 1, Mar. 1954, pp. 63-72

Author's Summary

Before the strengths of pozzolana-lime mixes can be used as a sensitive index of pozzolanic reactivity the mortars must be free from shrinkage cracking and should possess high strengths. It is shown that this can be achieved by working with tamped mixes of low water-cement ratio. Two methods for producing such mortars are described and the conditions for obtaining maximum strength and minimum scatter of results are defined. The practice of classifying pozzolanas on the basis of strength determinations made at a fixed pozzolana-lime ratio is shown to be subject to error. An alternative procedure is suggested.

The relationship between water-cement ratio and strength at low water contents is investigated. It is shown that at the lowest water contents the attainable strength increases linearly with the water-cement ratio,

the form of the relationship changing at higher water contents. Conditions for the preparation of mortars of similar physical characteristics are laid down on the basis of these observations. Such mortars should be used when comparing a series of pozzolanas. (See also "Pozzolanas—1. A Procedure for the Reproducible Determination of Pozzolonic Reactivity," "Current Reviews," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, p. 610.)

Effect of aggressive waters on cement and concrete, with particular reference to cement-lined mains

M. A. SIMMONDS, *Journal of the Institution of Engineers, Australia* (Sidney), V. 26, No. 1-2, Jan.-Feb. 1954, pp. 9-16

Discusses an investigation of the effects of aggressive water on cement and concrete. The investigation included the comparison of the cement linings of water mains before and after exposure to water from several areas. The investigations showed varying losses of cement matrix under exposure to the different waters. The sand-matrix ratio varied from 1.5 : 1 in unused lining material to as high as 12 : 1 in one case after an exposure of 11 years.

Two possible protective measures discussed are (1) treatment of the water and (2) treatment of the lining.

Handling of cement

DENNIS F. ORCHARD, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 2, No. 5, Sept. 1953, pp. 616-642

Reviewed by ARON L. MIRSKY

A thorough, well-illustrated study of various means of transporting, off-loading, and storing bulk cement (defined by author as any cement not packed in a container which, when full, can be lifted easily by hand).

System of lime, alumina, and water from 50 to 250 C

RICHARD B. PEPPLER and LANSING S. WELLS, *Journal of Research*, National Bureau of Standards, V. 52, No. 2, Feb. 1954

AUTHORS' SUMMARY

The system CaO-Al_2O_3-H_2O was investigated over the range 50 to 250 C. At equilibrium, only two stable ternary compounds exist in the system, namely, $3CaO \cdot Al_2O_3 \cdot 6H_2O$ up to 215 C and $4CaO \cdot 3Al_2O_3 \cdot 3H_2O$ from 215 to 250 C. Hexagonal hydrated calcium aluminates occur as metastable phases with decreasing stability with increasing temperature until at temperatures over 100 C they exist only momentarily. There are only two stable alumina hydrates, namely, gibbsite ($Al_2O_3 \cdot 3H_2O$) up to 150 C, and boehmite ($Al_2O_3 \cdot H_2O$) thereafter. A stable phase throughout the temperature range is $Ca(OH)_2$. The equilibra curves of these solid phases maintain the same general relationship to each other throughout. As the temperature increases to 250 C, the solubility fields are compressed into the water apex of the diagram.

Porosity determinations and the significance of pore characteristics of aggregates

D. W. LEWIS, W. L. DOLCH, and K. B. WOODS, *Proceedings*, ASTM, V. 53, 1953, pp. 949-962

AUTHORS' SUMMARY

This paper has been developed from material obtained from a review of the literature covering research data collected from the laboratory and the field. Historical aspects of the paper include references in the fields of building stone, brick, and aggregates used in portland cement concrete. Attention is directed toward various methods of test which have been employed for determining the characteristics of pores, and emphasis is given to the relationship of these characteristics to absorption and degree of saturation of aggregate and to the durability of concrete.

It is concluded that the abundance, shape, size, and continuity of pores influence the physical properties and the chemical stability of aggregates and control certain durability characteristics of portland cement concrete. It is concluded further that there is a definite need for research in connection with the development of methods of test for the determination of pore characteristics.

Significance of tests for chemical reactions of aggregates in concrete

WILLIAM LERCH, *Proceedings*, ASTM, V. 53, 1953, pp. 978-990

AUTHOR'S SUMMARY

Chemical reactions involving aggregates in concrete may be beneficial or deleterious depending on the nature of the reactions. Chemical reactions that increase the bond between aggregate and matrix may be beneficial, while those that cause abnormal expansion and consequent map cracking are deleterious. There are no direct tests to evaluate the beneficial reactions, but results obtained by some performance tests, together

with observations made in connection with the tests, give some indication of the bond between the aggregate and the matrix. Extensive studies have been made in recent years of the deleterious reactions of aggregates in concrete that can cause abnormal expansion. The American Society for Testing Materials has adopted three tentative methods of tests for aggregates to determine their potentiality for producing abnormal expansion in concrete. Additional methods of test have been described or are being studied. Some of these tests can be used also for the study of reactive siliceous admixtures or other inhibitors that may reduce or eliminate the expansion. This paper discusses the significance of these various tests for chemical reactions of aggregates in concrete.

Concrete aggregates

Technical Memorandum No. 6-370, V. 1-5, Waterways Experiment Station, Vicksburg, Miss., 1954

The Corps of Engineers, U. S. Army, requires that a survey be made to determine the properties of material locally and economically available that may be considered for use as concrete aggregates for its construction projects wherever they may be located. This aggregate survey work is conducted for each project by the division concrete laboratory nearest the site. These laboratories are located in Portland, Ore.; San Francisco, Calif.; Dallas, Texas; Omaha, Neb.; Vicksburg, Miss.; Atlanta, Ga.; and Cincinnati, Ohio.

Under the civil works investigations program sponsored by the Office, Chief of Engineers, the Waterways Experiment Station began in 1950 to compile on standard data sheets all of the information developed since 1946 in connection with aggregate surveys. The data which have been compiled to date cover 1210 sources of material located within the continental United States. A data sheet has been prepared on each of these sources. The United States has been divided into rectangles each one degree of longitude in an East-West direction and one degree of latitude in a North-South direction. A map of each of these rectangles has been prepared and the aggregate sources plotted on the appropriate map. The 1210 sources for which data are now available are located on 333 different one-degree maps. The United States has been divided into ten areas: Area 1 includes the Pacific Coast west of longitude 120; Area 2 includes the Northwest north of latitude 40, west of longitude 110; Area 3 is the Southwest, south of latitude 40 and west of longitude 110; Areas 4, 5, 6, 7, 8, and 9 are alternately north and south of latitude 40 and include the sections respectively west of longitude 100, 90, and 80. Area 10 includes that part of the United States east of longitude 80.

Although it is not feasible to issue these publications to private organizations and individuals, the five volumes of data may be examined, upon request, at any Corps of Engineers district or division office or laboratory. In this connection, the Waterways Experiment Station will maintain several complete sets of these volumes to be made available on a loan basis.

Prediction of concrete durability from thermal tests of aggregate

E. C. Higginson and D. G. Kretsinger, *Proceedings*, ASTM, V. 53, 1953, pp. 991-1001

Authors' Summary

Data are presented on the effect of freezing and thawing, wetting and drying, and other simulated weathering tests on various concretes containing either natural aggregate or aggregate composed of individual rocks and minerals. The effect of thermal coefficient and diffusivity of these aggregates on the deterioration occurring during these tests is discussed, but no correlation is found between the test results and the thermal properties of the materials. Evidence is presented showing that the expansion occurring in saturated concrete during the measurement of coefficient of expansion can be correlated with freezing-and-thawing test results, thus providing a quick method of predicting durability of concretes.

Apparatus for measuring moisture content of sand (Apparat for bestamning av vattenhalteni naturfuktig sand)

P. Hallstrom, *Betong* (Stockholm), V. 38, No. 3, 1953, pp. 161-167

Reviewed by Eivind Hognestad

Apparatus described in this paper does not necessitate weighing the sand and is believed to be better suited for field work than other methods. Water content is determined by measuring the difference between the amounts of water needed to saturate moist and dry sand. Maximum error is about ± 0.5 percent water content.

Evaluation of curing compounds for portland cement concrete

C. E. Proudley, *Proceedings*, ASTM, V. 53, 1953, pp. 1069-1078

Author's Summary

Desirable characteristics of curing compounds for use on highways and structures are enumerated and laboratory tests simulating field conditions for the purpose of evaluating liquid, membrane-forming compounds are described. Factors affecting the correlation between laboratory tests and service performance are discussed as are the difficulties encountered in securing uniformity among testing laboratories under the present methods. Influence of job operations and climatic conditions upon the performance of curing compounds is touched upon briefly.

Thermal expansion tests on aggregates, neat cements, and concretes

Leonard J. Mitchell, *Proceedings*, ASTM, V. 53, 1953, pp. 963-977

Describes apparatus used to measure accurately length changes in small test specimens. Test data are reported on 41 aggregate minerals, 11 portland cements, 2 portland-pozzolan cements, and many non-air-entrained and air-entrained concretes. Other variables investigated were moisture content, mix proportions, air content, and test methods. Author concludes that coefficient of thermal expansion of concrete can be determined from the weighted averages of the coefficient of thermal expansion of the various constituents if the correct moisture effect can be determined.

Pavements

Contraction joints sawed on Virginia test road

Contractors and Engineers, V. 51, No. 3, Mar. 1954, pp. 58-59

In Virginia's tests of sawed joints the 0.924-mile test road was divided into three sections with Section 1 containing joints sawed into the continuously wire-mesh-reinforced pavement, Section 2 containing joints sawed over dowel bar assemblies, and Section 3 containing standard formed dummy joints over dowel bar assemblies. The sawed joints were ⅛ in. wide and 1¾ to 2 in. deep and cost about $12.50 per 24-ft joint. The test also considered joint spacing, time between placing and cutting, and relative pavement roughness between standard dummy joints and sawed joints.

Concrete resurfacing of concrete pavement in various stages of deterioration

D. E. Gotham and George W. Lord, *Bulletin* No. 87, Highway Research Board, Jan. 1954, 44 pp., $0.60

Describes construction and performance of two experimental sections of concrete resurfacing of concrete pavement built in Missouri, the first in 1932 and the other in 1936. The part constructed in 1932 used two thicknesses, 4 in. and 6 in., each covering about ¼ mile and purposely located to cover old pavement with subsections in various stages of deterioration. The 11¾-mile section constructed in 1936 was placed in three thicknesses, 4 in., 5 in., and 6 in., according to the condition of the old pavement.

With the aid of an accurate record kept on the old pavement, a correlation of the defects of the old and new pavements was made. Evaluation of the performance of the resurfaced concrete attempted to identify the effects of (1) thickness of resurfacing, (2) condition of old pavement, (3) drainage, (4) expansion joints, (5) cross section design, and (6) "marker seal" center joint.

Appendices include (1) effects of old pavement condition on the condition of resurfaces based on study of individual effects, (2) a study of economic considerations in the selection of resurfacing thickness, and (3) a method of spacing transverse joints in concrete resurfacing.

New equipment is used to pave parking apron

Ray Day, *Contractors and Engineers*, V. 51, No. 5, May 1954, pp. 44-47

Describes the equipment used to pave the odd-shaped apron, 4175 ft long, 1175 ft wide at broadest section, and 700 ft wide at narrowest section, at Davis-Monthan Air Force Base. The series of 25-ft lanes are joined together by formed keyways with dowel steel bars at the outside lanes and construction joints only.

German and foreign concrete mixers for concrete pavement construction (Deutsche und auslaendische Betonmischer fuer den Betonstrassenbau)

W. E. Fauner and H. Porsch, *Zeitschrift VDI* (Duesseldorf), V. 95, No. 32, Nov. 11, 1953, pp. 1100-1103

Reviewed by Anon L. Mirsky

A comparison of German, American, and French pavers. Although principle is the same, the various mixers differ in constructional details.

Concreting the world's longest runway

Engineering News-Record, V. 152, No. 12, Mar. 25, 1954, pp. 44-46

Reviewed by S. J. Chamberlin

The 15,000-ft runway at Edwards Air Force Base in California is 17 and 19 in. thick. The 5.88-sack concrete mixture with 3 in. maximum size aggregate, 4 percent entrained air, and 1½ in. slump, is designed for a 90-day flexural strength of 620 psi. Two pavers place a 25-ft width and are followed by a spreader with six internal vibrators. The mechanical finishing machine includes two transverse vibrating screeds 15 ft apart and is followed by a mechanically operated longitudinal float. A 16-bladed saw cuts contraction joints at 25 ft, ⅛ in. wide and 4 in. deep. Joints were cut at 24 to 26 hr during the winter but it is expected that it can be done at 8 hr during warmer weather.

Divided highway paved with 10-in. concrete

Contractors and Engineers, V. 51, No. 3, Mar. 1954, pp. 16-20

Describes the pavement and construction procedure for the modernization of U. S. 66 in Illinois between Chicago and Springfield. The pavement is 24 ft wide and 10 in. thick, one of the heaviest in the area.

Limited access highway paved with reinforced concrete on slag subgrade

Contractors and Engineers, V. 50, No. 10, Oct. 1953, pp. 16-19

Step-by-step account of grading and paving, including types of equipment. The 10-in. reinforced concrete slab was laid on a special subgrade of slag 6 in. deep under the inside edge and carried out to 8 in. deep under the shoulder which was 10 ft wide and composed of stabilized slag.

Test road is paved with 9-in. concrete

Contractors and Engineers, V. 51, No. 4, Apr. 1954, pp. 82-85

Describes pavement and paving operations on test portion of Indiana's U. S. 31.

Automatic batch plant speeds airfield paving

Albert C. Smith, *Contractors and Engineers*, V. 51, No. 2, Feb. 1954, pp. 8-11

A new operational apron at Lockebourne Air Force Base containing nearly 300,000 sq yd of concrete pavement was laid in 4 months. To maintain a high production rate an automatic batching plant was used. Mixing a blend of 80 percent portland and 20 percent natural cement with sand and two sizes of stone the plant maintained 130 batches per hr.

Concrete paving job done in fast time

Contractors and Engineers, V. 50, No. 12, Dec. 1953, pp. 94-98

Describes the equipment and methods used in completing a 5.2 mile stretch of divided, four-lane highway in 55 days.

South Lille motorway (L'autoroute sud de Lille)

Bulletin, Societe des Ingenieurs Civils de France (Paris), No. 16, 1953, pp. 248-254

Reviewed by Aron L. Mirsky

Description of a 23-mile stretch of 100-mph superhighway between Paris and Lille. Divided roadways will be of bituminous material on macadam laid on a concrete base; latter, about 12 in. thick, will be of blast furnace slag cement concrete.

Precast concrete

Manual of precast concrete construction

F. Thomas Collins, San Gabriel, Calif., 3rd Edition, 1954, 102 pp.

Methods and procedures are shown for general precast concrete construction. Most of material is concerned with tilt-up construction, but thin-shell precast concrete construction is also covered. This third edition differs from the earlier manual (see "Current Reviews," ACI Journal, Sept. 1953, *Proc.* V. 50, p. 102) in that the data has been brought up to date, most of the text rewritten, material added on prestressing of wall panels, and a new chapter added on sandwich wall panel construction.

Thin-shell precast concrete framing speeds warehouse construction

Contractors and Engineers, V. 50, No. 10, Oct. 1953, pp. 108-112

The construction of three warehouses, each of which is almost entirely precast, is described. The two basic applications of thin-shell precasting are ribbed panels and hollow-section frames. Ribbed panels are used for the roof and walls and the hollow sections for the rigid frames. The voids in the hollow sections were made by placing the concrete around fiberboard tubes. The frame components are designed so that the splices occur at points of inflection.

New method of lining a tunnel

Concrete and Constructional Engineering (London), V. 49, No. 4, Apr. 1954, pp. 141-145

Describes a 1000-ft experimental tunnel constructed of precast concrete rings with 7 ft 6 in. internal diameter and an inner lining of steel coated with fine concrete. The space between the steel inner lining and the precast concrete lining was filled with a grout of sand and cement, through 3-in. diameter threaded holes provided for the purpose. The steel pipes were in lengths of 14 ft 6 in. with an external diameter of 6 ft 6 in. The inner surface of the pipes was lined with spun concrete.

The precast segments formed a complete ring of lining which bore tightly on the clay. The segments were formed with the outside arc at one end 5 in. longer than at the other end. These wedge-like segments were placed with the large ends alternately outward. When all ten of the segments in a ring were properly oriented, these alternate segments were forced into position forming a rigid ring with all edges flush.

Basalt introduces the precast concrete home

Rock Products, V. 57, No. 1, Jan. 1954, pp. 132-135

Precast concrete houses produced by the Basalt Rock Co. are described. A step-by-step description of the manufacture and erection of these complete houses is included.

Prefabrication (Prefabricacion)

Jose M. Eymar, *Informes de la Construccion* (Madrid), No. 57, 1954, Item No. 832-11

Describes methods of making, curing, and placing precast building products including wall and ceiling tile, window and door frames, sunshades, and ducts for air conditioning.

Roofing costs are cut in plant construction

H. S. Freeman, *Contractors and Engineers*, V. 51, No. 4, Apr. 1954, pp. 66-67

Handling precast roof slabs according to a set plan cut costs to about six cents per slab.

Prestressed concrete

Prestressed concrete (Le beton precontraint)

Gustave Magnel, 3rd Edition, Editions Eecheyr, Ghent, Belgium, 1954, 511 pp., $12.50

Reviewed by Charles C. Zollman

This third edition, which is the fourth volume of the French language series "Design and Analysis of Reinforced Concrete," includes various sections, substantially expanded, which were presented in previous editions.

Various prestressing methods not previously described are incorporated in the first section.

In the section on design and analysis of simply supported beams, the author has now included a number of typical beam cross sections for various spans, depths, and loads. A new chapter on ultimate design of prestressed beams is also presented. A new approach to the problem of end-blocks has been established on more reasonable assumptions.

The sections on design and analysis of indeterminate structures have been completely revised and include the design of several continuous girders and rigid frames. Considerations on the economy and failure loads of continuous beams are given.

The section of field and laboratory tests has been considerably expanded. It includes the field test of the 160-ft span Walnut Lane girder, the laboratory test to failure of a full size 55-ft girder, tests of partially prestressed beams, of a continuous three-span girder having a total length of 60 ft, a 10 x 10-ft slab resting on four supports, tests of prestressed poles, static and dynamic tests of various beams, fire tests, and tests of prestressed beams exposed to extreme cold.

Construction procedures adopted for numerous projects of all types and erected in various countries such as Belgium, Switzerland, United States, Great Britain, and Holland are described in the vastly expanded section on prestressed concrete applications.

A section on design and analysis of prestressed structural steel framing has been added. The laboratory test of a truss approximately 50 ft long and 3 ft deep, is presented in detail. A brief description of the construction of a prestressed structural steel aircraft hangar 450 x 200 ft with one interior column completes this latest application of the concept of prestressing.

It is this reviewer's opinion that the book is unique, inasmuch as the various aspects of prestressing are so logically interwoven. Design and analysis are first developed, then checked by field and laboratory tests of large, full-size elements, and it is only there-

after that the construction of bold and daring projects is described. A wealth of information has been made available to the engineer under one cover.

Design of simply supported prestressed concrete beams for ultimate loads

FREDERICK W. GIFFORD, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 3, No. 1, Apr. 1954, pp. 125-143

Reviewed by ARON L. MIRSKY

Author develops an ultimate-load theory for beams of rectangular or flanged (I) section, and introduces the safety-factor concept by combining resulting equations with his equations for working loads (*Proceedings*, Institution of Civil Engineers, Part III, V. 2, No. 3, Dec. 1953, pp. 589-603; "Current Reviews," ACI JOURNAL, Sept. 1954, p. 108). Experimentally found values of the ultimate load coefficients are discussed and recommendations for values to be used in design are given; assuming the beams to fail by crushing of the concrete (the more usual case), author presents a simple method, based on the stress-strain curve of the prestressing steel, for finding the steel stresses if the factor of safety is specified, or the factor of safety if the steel stress is given.

Prestressed concrete in highway bridges and pavements

CURZON DOBELL, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 188-208

Brief historical review of use of prestressing in highway structures and pavement. Methods of prestressing, types of prestressed bridges, and costs are covered.

Corrosion of steel in prestressed concrete

Concrete and Constructional Engineering (London), V. 49, No. 6, June 1954, pp. 201-202

Brief description of tests to determine the resistance to corrosion of steel in prestressed concrete. Specimens used were of a 1: 1½ : 3 mix using high-alumina cement. Twelve high-tensile steel wires of 0.079-in. diameter were used in the 2-in. x 6-in. x 9-ft 6-in. specimens.

Why should we use prestressed concrete?

R. F. SHAW, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. A-1–A-10

Asks some thought-provoking questions about the design, construction, durability, and cost of prestressed concrete construction.

Partially-prestressed reinforced concrete helical stairs

Concrete and Constructional Engineering (London), V. 49, No. 6, June 1954, pp. 196-198

Brief description is given of partially-prestressed reinforced concrete helical stairs used in recent extensions of Bologna University.

Fire resistance of prestressed concrete

A. W. HILL, *Constructional Review* (Sidney), V. 26, No. 10, Feb. 1954, pp. 29-38

A discussion is given of tests, carried out in Australia, on the fire resistance of prestressed concrete elements. Included are the equipment used, the procedures followed, and the results obtained.

Experience with bridge prestressing methods

Engineering News-Record, V. 152, No. 11, Mar. 18, 1954, pp. 38-42, 46

Reviewed by S. J. CHAMBERLIN

A recently completed experimental project by the Massachusetts Department of Public Works consisted of four bridges with the concrete beams post-tensioned by: (1) strands with swaged fittings held by nuts; (2) bars anchored by nuts; (3) wires with pressed-on heads; and (4) wires gripped by conical wedges. Labor requirements are tabulated and costs of the beams are given. All beams have an I-shaped cross section, were cast in an outdoor yard, and have a cast-in-place deck. Most of the difficulties were of a minor nature.

Results from a questionnaire to other state highway departments are summarized in a table of bridge superstructure costs listing (1) the name; (2) location; (3) year; (4) spans; (5) method of prestressing; and (6) cost per sq ft. More significant than the listed costs is that in many cases bids were lower than those for alternate designs.

Elimination of moments in shell roofs by prestressing

WILLIAM T. MARSHALL, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 3, No. 1, Apr. 1954, pp. 276-282

Reviewed by ARON L. MIRSKY

Author demonstrates that while, in general, secondary stresses due to deflection of the loaded shell cannot be completely eliminated by prestressing, it is possible to eliminate the transverse bending moments by such means. Final stresses at all points will then be equal to the membrane stress plus a constant, this

latter being equivalent to the stress due to a load applied horizontally along the axis of the shell and thus causing no transverse moment. A numerical example is given.

Long-span prestressed concrete beams used in Army field house

ROBERT E. SNETZER, *Civil Engineering*, V. 24, No. 4, Apr. 1954, pp. 41-43

Describes the casting and erection of 22 prestressed beams for an Army field house to meet a clear opening requirement of 100 x 180 ft with a 20-ft minimum height. The beams were cast and prestressed at the site. They were positioned with two stiff-leg cranes on rails and a specially designed bracing truss, which was required because of the extreme limberness of the beams. Each beam is 106 ft 6 in. long and 5 ft 3 in. deep at the center.

Composite prestressed beams and in-situ slabs

ALAN H. MATTOCK, *Concrete and Constructional Engineering* (London), V. 49, No. 4, Apr. 1954, pp. 123-132

Discusses design of a composite system of a prestressed beam and a cast-in-place slab that act together when the slab is hardened. Diagrams and design calculations are given for two cases: (a) the beam supported at intervals while the slab is placed, and (b) the beam not supported during placing of the slab.

German recommendations for prestressed concrete

Concrete and Constructional Engineering (London), V. 49, No. 6, June 1954, pp. 189-192

Presents recently issued German recommendations for design and construction of prestressed concrete. Included are tables on (1) properties of steel and concrete, (2) coefficients of creep and shrinkage, (3) permissible stresses, and (4) permissible tensile stresses in concrete in bridges carrying normal railway traffic.

Control of concrete quality in prestressed construction

R. B. YOUNG and J. N. MUSTARD, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. D-1 D-5

Concrete used in prestressed construction is essentially the same as that used in other work. Any differences are of degree, not of kind, and the same principles govern its formulation, production, handling, and placing. This paper is not limited to concrete used only for prestressing, but deals with the fundamentals of its control regardless of use.

Tacoma plant features factory prestressing

Concrete, V. 61, No. 9, Sept. 1953, pp. 3-6

Describes plant fabrication of precast prestressed units. Construction of the pilot plant itself utilized prestressing in practically all structural elements: waffle window casings, T-section columns, girders, roof purlins, and roof deck; wall panels were of precast concrete.

Properties of concrete

Comparison of dynamic methods of testing concretes subjected to freezing and thawing

G. M. BATCHELDER and D. W. LEWIS, *Proceedings*, ASTM, V. 53, 1953, pp. 1053-1068

AUTHORS' SUMMARY

The results of resonant frequency and velocity (Soniscope) tests on concrete mixes of varying durabilities when subjected to freezing and thawing are discussed. Changes in velocity squared and in modulus of elasticity calculated from resonant frequency are compared. For two mixes, the tests included dynamic determinations of Poisson's ratio. Poisson's ratio values so obtained were used to calculate modulus of elasticity values from velocity, and these were compared with the values obtained from resonant frequency tests.

It was concluded that, for the concretes tested, the velocity tests were less sensitive than resonant frequency measurements as a measure of the deterioration of the specimens. No advantage was found in calculating modulus of elasticity values from the velocity measurements; change in the velocity squared was a better measure of deterioration. Differences between the changes in these two quantities were caused by apparent changes in Poisson's ratio during the freezing and thawing of nondurable concrete.

Significance of tests for sulfate resistance of concrete

E. C. HIGGINSON and O. J. GLANTZ, *Proceedings*, ASTM, V. 53, 1953, pp. 1002-1020

Many investigators have made field and laboratory studies of sulfate attack on concrete. This paper summarizes the various test methods used and the results obtained to date.

Determination of compressive strength of concrete by using its sonic properties

CLYDE E. KESLER and YOSHIRO HIGUCHI, *Proceedings*, ASTM, V. 53, 1953, pp. 1044-1052

AUTHORS' SUMMARY

In predicting the quality of concrete by sonic methods, the prediction is usually based on the initial modulus of elasticity; however, considering concrete as viscoelastic, an analysis of a rheological model of concrete clearly shows that a property in addition to the initial modulus of elasticity must be considered. The one property, other than modulus of elasticity, that promises to be of greatest help in predicting the strength of concrete is the coefficient of viscosity. While this property may be an excellent one to use, it cannot be determined readily from sonic tests of concrete; but, logarithmic decrement, which is related to the coefficient of viscosity, can be easily obtained from sonic tests.

To establish whether or not there exists a relationship between the modulus of elasticity and damping capacity of concrete and the compressive strength of concrete, some 300 standard 6 x 12-in. cylinders were tested. From the test data a set of curves is obtained from which the strength of concrete, made of aggregates used in this investigation, can be predicted. The accuracy of the prediction is generally within an error of 5 percent. This accuracy can be obtained without knowledge of the age, mix, or moisture content of the concrete. The results clearly show why the use of the modulus of elasticity alone is not sufficient to predict accurately the strength of concrete.

Contribution to the study of fresh concrete. Application to the analysis of the vacuum concrete process (Contribution a l'etude du beton frais. Application a l'analyse du traitement vacuum concrete)

M. LEVIANT, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 2, Mar.-Apr. 1953, pp. 163-184; No. 3, May-June 1953, pp. 315-356; No. 4, July-Aug. 1953, pp. 461-479

Reviewed by ARON L. MIRSKY

Starting with a discussion of the paste or mortar, paper proceeds to a study of the action of fresh concrete under compaction with simultaneous drying (removal of a portion of the mixing water). Similarity of compaction process to that of soils is made use of (although author notes that the theory of compaction of fresh concrete is more complex); it is also noted that the law of penetration of vacuum in fresh concrete is similar to Fourier's law of heat penetration in a wall. Conclusion: vacuum treatment results in (1) shrinkage of the aggregate skeleton, (2) reduction of water content with consequent formation of a "microskeleton" of cement particles, (3) enhanced cohesion or "pseudo-hardening" permitting immediate removal of forms, and (4) increased strength of the cured concrete due to reduced W/C ratio.

Bibliography on sulfate resistance of portland cements, concretes and mortars

DALTON G. MILLER, PHILIP W. MANSON, and ROBERT T. H. CHEN, *Paper* No. 708, Miscellaneous Journal Series, University of Minnesota, 1953, 134 pp.

Annotated bibliography of more than 400 references arranged alphabetically by author. While the action of sulfate waters and soils on concrete was the principal reason for inclusion of references in this bibliography, considerable space has been given also to articles dealing with the action of sea water, acids, and other deleterious agents. At the end of the abstracts, a selected list of papers with noteworthy bibliographies is included.

Alkali-aggregate phase of chemical reactivity in concrete

J. A. HESTER and O. F. SMITH, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 306-316

The progressive reaction of alkali and aggregate in concrete depends on lime to liberate the alkali from the alkali silicates before continuing the cycle. To support this theory, results of a chemical study of this reaction cycle are given. Results from the analysis of the concrete gel are in accordance with progressive-reaction theory.

Analysis of mortar bars disclosed little or no alkali silicates. The presence of free alkali, near the original percent, dispelled the possibility that the alkali silicates had escaped. The acid-soluble silica in the mortar bars varied directly with the alkali content and total expansion. The cement showing the highest expansion in the mortar bars contained the highest magnesia as well as the highest alkali content.

Wear tests of concrete

H. L. KENNEDY and M. E. PRIOR, *Proceedings*, ASTM, V. 53, 1953, pp. 1021-1032

AUTHORS' SUMMARY

A brief history of early tests used to evaluate the resistance of road materials and concrete to abrasion is presented. The types

of wear generally experienced on concrete surfaces are broken down into four general categories and the causes discussed in each case. Methods of test used by investigators to evaluate the wear resistance of concrete are discussed for the various types of wear. Advantages and disadvantages of the test procedures have been considered and comments made regarding the proper type of test for a particular kind of wear. The effect of such factors as strength and curing of the concrete have been included in the discussion, since they have an important bearing on the ability of the concrete to resist wear.

High-strength concrete

The Engineer (London), V. 197, No. 5134, June 18, 1954, p. 902

Reviewed by Aron L. Mirsky

Brief summary of some recent studies on high-strength concrete by the Cement and Concrete Assn. For 28-day strengths above 6000 psi, type of aggregate, aggregate/cement ratio, and workability were found significant in determining strength. Gap grading and steam curing are also discussed.

Investigation of durability of concrete exposed to natural weathering—Report No. 5, summary of results 1936-1953

Technical Memorandum No. 6-226, Waterways Experiment Station, Vicksburg, Miss., May 1954

Brief description of natural weathering stations and test procedures at Treat Island, Me., and St. Augustine, Fla., operated by the Corps of Engineers. Twenty investigational programs are summarized. A bibliography gives references to more detailed published findings from these tests.

Permeability and triaxial tests of lean mass concrete

Technical Memorandum No. 6-380, Waterways Experiment Station, Vicksburg, Miss., Mar. 1954

Laboratory cylinders of 15 low-cement concrete mixtures were tested to determine permeability under high hydrostatic pressures encountered in dams. Laboratory data were verified by specimens taken at five construction jobs at dams and by 10-in. cores from Arrowrock Dam. Other specimens were tested under triaxial loads. Each 15 x 30-in. cylinder contained a Carlson strain meter to observe axial strains due to hydration of cement, saturation of the specimen, lateral pressure, pore pressure, and externally applied load.

Field Soniscope tests of concrete

Technical Memorandum No. 6-383, Waterways Experiment Station, Vicksburg, Miss., Apr. 1954

Reports pulse velocities measured at selected points on a number of structures. Brief description of Soniscope and its use. Appendix gives proposed tentative method of test for the measurement of the pulse velocity of propagation of elastic waves in concrete.

Variations of concrete strength in the light of Soviet and our investigations (in Czech)

K. Hruban and V. Hlavacek, *Bulletin of the Brno Institute of Constructional Engineering and Architecture* (Czechoslovakia), V. 1, No. 14, 1952, 14 pp.

Reviewed by Ivan M. Viest

This is a review of a Russian statistical study of tests on over 30,000 concrete cubes reported by S. I. Minc in Moscow in 1949 and a similar study of tests on 428 cubes and 104 modulus-of-rupture beams made in Czechoslovakia. The following problems are studied: concretes manufactured in central mixing plants vs. those mixed on the job; significance of a test of three cubes; effect of age on standard deviation; compression tests vs. bending tests.

Structures

Corrugated concrete shell roofs

James H. deW. Waller and Alan C. Aston, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 2, No. 2, Aug. 1953, pp. 153-196 (including oral discussion)

Reviewed by Aron L. Mirsky

On the banks of the Tigris, some 15 miles from Bagdad, stands the great arch of Ctesiphon, a vaulted structure with a rise of 112 ft and a span of almost 90 ft, constructed of mud bricks set in Tigris mud. A visit to it inspired Mr. Waller, author of the first part of this paper (pp. 154-171), to a 30-year study of Ctesiphon-type structures in concrete, leading to the conclusion that, with a working stress of 1200 psi, an arch of 700-ft span and 140-ft rise is entirely possible in plain concrete, and the thickness of the shell would not exceed 4½ in. Such shells are inherently waterproof ("Concrete does not 'always crack.' In pure compression it never 'cracks' though it may crush or be affected by tension derived from shear."), possess natural air-conditioning, and author speculates on the possibilities of constructing a large shell to cover an entire neighborhood, the individual buildings of which could then be of light construction, since protection

from the elements would be afforded by the enveloping shell.

In the second part, titled "The Theoretical Aspect of Corrugated Concrete Shell Arch Roofs of Catenary Form" (pp. 171-179), Mr. Aston discusses the stresses in catenary arches, the advantages of corrugated *vs* ribbed arches, and the design factors of importance. An appendix (pp. 179-182) summarizes the design of five proposed aircraft hangars (one 306 ft long, two 120 ft long, and two 200 ft long) at Lara, Australia, which are to have a span of 310 ft and a rise of 64 ft. There is to be some steel reinforcement, amounting to 2 lb per sq ft of floor area: first, to take some tensile stress; second, to act as temperature and shrinkage reinforcement; and third, to act as a most important part of the supporting structure for the pneumatically-placed concrete.

An excellent summary of this paper may be found in *Engineering* (London), Feb. 13, 1953, p. 211.

Guildhall: Provision of new foundations to the north wall by underpinning and other means

Bernard Geen, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 3, No. 2 Mar. 1954, pp. 210-217

Reviewed by Aron L. Mirsky

London's Guildhall has had a long and distinguished past, commencing with its construction in 1411. An air raid in 1940 resulted in extensive damage to the roof and to buildings on the north side of the hall. During construction of a temporary building on that side in 1951, excavation disclosed disturbing foundation conditions which led to the decision to provide better support for the buttressed north wall.

Reinforced concrete beams resting on piles were constructed on either side of each buttress. The buttresses were then squeezed as in a vise, the beams forming the jaws, by prestressing cables passing through the buttresses and anchored in the beams. This method necessitated no excavation below the existing foundations, and resulted in a minimum of weakening of any buttress during operations; the levels of the supporting beams could be selected to suit the local foundation elevations; and each new foundation became effective immediately on completion, independently of the status of operations on any other buttress.

Steel forms are used on big powerhouse

Ray Day, *Contractors and Engineers*, V. 51, No. 2, Feb. 1954, pp. 22-25

Thirty-two-ft steel panel forms are being used on the powerhouse of Chief Joseph Dam. The size and scope of the concrete work favored these forms which permit placing up to 70 ft of concrete at one time and can be reused. Also describes concrete production setup and placing cranes.

Vacuum processing paces double-deck highway job

Albert C. Smith, *Contractors and Engineers*, V. 51, No. 4, Apr. 1954, pp. 10-14

By vacuum processing the upper deck of a double-deck highway the contractor was able to strip a traveling form from underneath in 2½ days. Ordinarily such a heavy deck could not have been stripped for about 14 days. The procedure followed from setting forms to placing concrete is described.

Concrete farm structures

A. M. Pennington, Concrete Publications, Ltd., London, England, 1954, 154 pp., $2.80

Deals simply and practically with design and construction of precast concrete farm buildings, generally of block construction. It gives plans, sections, working drawings, and construction details for cow barns, dairies, stables, liquid manure tanks and dung pits, implement sheds, silos, culverts, water troughs, gates, yards, and roads. Information is given on the manufacture of the precast units. Designs are in accordance with British practice, and much of the material is also in accordance with American practices.

Reconstruction of the ground floor of the Royal Edward cold store, Avonmouth Docks

William J. Sivewright and Stanley P. Whittington, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 3, No. 1, Apr. 1954, pp. 166-182

Reviewed by Aron L. Mirsky

Frost-heave damage to the ground floor of this cold storage plant became apparent three years after it was built (1922). Plant is a four-story concrete-frame structure, 103 x 200 ft in plan, with brick walls. Original ground floor consisted of 6 in. of unreinforced concrete laid directly on 12 in. of ashes underlain by about 50 ft of soft silty clay; insulation consisted of two 3 in. layers of granulated cork, topped with a 2-in. granolithic wearing course.

New cold storage floor is of suspended-type, precast concrete sections topped with 3 in. of cast-in-place concrete, placed on a grid of concrete-encased steel beams spanning between existing pile caps. New insulation is 8 in. thick; wearing surface is 3 in. (surface in passageways and other areas subject to much traffic is 1 in. of East African hardwood nailed to sleepers embedded in the concrete). The ground beneath the new floor was excavated to provide an air and crawl space. Elevator pits also required special attention in design and reconstruction.

A short appendix deals briefly with the effect of reinforced concrete piles on ground temperatures beneath a cold storage plant having an air space below the cold rooms.

General

Making durable concrete drain tile on packer-head machines

PHILLIP W. MANSON and DALTON G. MILLER, *Bulletin* 426, University of Minnesota Agricultural Experiment Station, May 1954, 16 pp.

Discusses necessity for well-graded sound aggregates and sufficient cement, maximum of mixing water, ample compaction of the concrete, and adequate curing to the production of acceptable concrete drain tile on packer-head machines. Appendix summarizes Iowa State College tests on bedding and allowable drain tile depths.

Two-way radio in Columbus ready-mix plant

Concrete, V. 61, No. 9, Sept. 1953, pp. 12, 14, 32

Describes the use of two-way radio in a commercial ready-mixed concrete operation.

Japanese scientific terms—Civil engineering

Japan Society of Civil Engineers (Tokyo), 400 pp., 1954

A two-part Japanese-English and English-Japanese dictionary of civil engineering terms giving Japanese words and pronunciations and the English equivalent. The book apparently is intended principally for Japanese use since all explanatory matter is in that language and a knowledge of the Japanese written language appears essential to intelligent use of the dictionary sections.

Billboards made of concrete masonry units

Concrete, V. 61, No. 9, Sept. 1953, p. 26

A unique use for concrete masonry units.

Mixed-in-place piles form jetty to control beach erosion

N. L. LIVER, E. C. MARDORF, and J. C. KING, *Civil Engineering*, V. 24, No. 3, Mar. 1954, pp. 56-57

Describes an experiment carried on in Ohio where an erosion control jetty was constructed, in Lake Erie, of mixed-in-place grout piles. The 24-in. piles were drilled into a sand fill dredged from the lake bottom. The final jetty was a continuous wall 96 ft long of overlapping piles 20 in. on center.

Recent wharves and jetties in France

Concrete and Constructional Engineering (London), V. 49, No. 4, Apr. 1954, pp. 133-139

Describes several wharves and jetties constructed in France since 1948 in which large precast elements were used. Diagrams of important details are included.

Formwork evolution (Evolution des coffrages)

L. J. KAHN, *Supplement to the Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 77, May 1954, Series: Concrete—Reinforced Concrete (27), 16 pp.

AUTHOR'S SUMMARY

After reviewing the various forming procedures used in recent years in France, the author describes some of the forming techniques used in the United States, particularly the use of plywood. A test program just started is outlined. Problems to be investigated are kind of wood, thickness, glues, protection of the faces and edges, standardization, pressure of fresh concrete, stiffeners, reuse, form removal, joints, vibration, and modulation.

Manufacturing processes for a new cement plant

BERNARD ULRICH, *The Engineering Journal* (Montreal), V. 37, No. 6, June 1954, pp. 684-687, 702

Reviewed by ARON L. MIRSKY

New $13,000,000, 1,500,000 bbl per yr, wet-process plant being built at Villeneuve, Quebec, Canada, embodies several unique design features, particularly as regards homogenizing the slurry, recovering dust and reintroducing it into the manufacturing process, and placing the storage silos above the packing house to take advantage of gravity flow. Technical and financial background of project has been provided by a Swiss concern, which sent a group of young engineers to Canada; this group, with experience on European cement plants, coordinated European ideas with Canadian conditions and

methods to produce an efficient and interesting plant. The thinking underlying the new features is well presented, illustrated with descriptions of some recently completed plants in various countries.

Elevated water tanks (Vattentorn, vattentekniska synspunkter)

S. Hesser, *Betong* (Stockholm), V. 38, No. 3, 1953, pp. 169-181

Reviewed by Eivind Hognestad

Paper discusses the purpose of water tanks and methods of estimating the storage volume and pressure needed. Various types of water tanks are described.

Field testing of concrete—New and simplified methods for testing concrete and its aggregates

Sven Thaulow, Norsk Cementforening, Oslo, Norway, 1952, 64 pp. (abridged English version of Norwegian original)

Reviewed by Paul Klieger

Author has directed his efforts during the past ten years toward developing new or simplified test procedures and combining them into a method of concrete control suitable for use at most building sites. Author states that emphasis has been placed on making the testing equipment as sturdy and simple as possible and the test methods simple and reliable to a point where "operators' idiosyncrasies" are reduced to a minimum.

The report describes the testing equipment and test procedures finally selected in considerable detail. From the emphasis placed on unit weight determinations of aggregates it appears that volumetric proportioning of concrete ingredients is still widely used in Norway. The testing of the fresh concrete includes determinations of consistency by a "concrete tester" which appears to be a simplified version of the Powers' remolding apparatus. This concrete tester is a versatile component of the author's concrete control kit. In addition to consistency determinations, a portion of the concrete tester is used for determining unit weight, specific gravity and moisture content of aggregates, unit weight and air content of fresh concrete, and composition analysis of the fresh concrete by various methods. (See also "News Letter," ACI Journal, Mar. 1954, *Proc.* V. 50, p. 10, for description of equipment.)

Testing procedures for the hardened concrete are described and parallel closely current practice in this country, except for the type of cylinder mold in use. The steel mold described in this report has end plates at both ends. After casting, the top end plate is fixed and the mold and specimen are then stored horizontally. No capping is required prior to testing.

Railroad rails on concrete slabs without ballast (in German)

O. Emmerich, *Zeitschrift des VDI* (Duesseldorf), V. 95, No. 4, Feb. 1, 1953, pp. 101-105

Placing of railroad rails on concrete slabs without ballast (in French)

Genie Civil (Paris), V. 130, No. 19, Oct. 1, 1953, pp. 374-375

Reviewed by M. W. Jackson

The second article translates most of the first one which is a general discussion of the problems of railroad tracks on slabs, such as concrete bridge decks without ballast. Ballast ordinarily gives elastic support to the rails. Without ballast it is necessary to design a flexible support so as to reduce the "dynamic coefficient" due to the passage of trains. Generally the connections of the rail to concrete contain a rubber pad; the original article shows five types, and paper includes seven references.

Concrete products yearbook, 1954 (Betonstein Jahrbuch, 1954)

Bauverlag GmbH, Wiesbaden, Germany, 1954, 432 pp., 5 DM

Reviewed by Aros L. Mirsky

Although the general format of this yearbook follows that of the 1953 edition ("Current Reviews," ACI Journal, June 1953, *Proc.* V. 49, pp. 979-980), many of the topics and of course the statistics are new.

Part A summarizes the organization of the industry, including the various technical groups, and gives latest (1953) statistics. Part B contains short articles on such subjects as aggregate particle sizes; coloring of concrete; reinforcement; admixtures; machinery for mixing, placing, compacting, and handling concrete; and the significance of materials testing. Part C, besides abstracts of German standards pertaining to concrete structures and products, has short articles on concrete roofs and slabs, chimney units, stairs, sources of trouble in concrete construction, pipes, and lighting and railway standards (masts). Part D contains tables of freight rates, a table of steel areas, and the index.